THE RAYMOND COMPOSITE PILE

BY

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OUTLINE

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I. OBJECT OF THESIS

The author's object in writing this thesis on Raymond Composite Piles is to gather together and present in its chronological order the mass of information available on the subject and to discuss this type of piling with respect to its uses, its installation, its strength, and its practicability. When referring to this thesis, the reader must bear in mind the fact that at the present time this subject is practically a new field in foundation work. Various tests and experiments on composite piles are still being conducted, and, although the pile as now being installed, serves acceptably the purpose for which it is intended, the composite pile of ten years hence will probably differ considerably from the present one.

II. HISTORY OF SUBJECT

*"Prior to the beginning of the present century, piling for foundation purposes, as used in this country, was almost invariably of wood. It is true, of course, that special conditions had at times imposed the necessity of introducing substitutes for

*History and Present Status of the Concrete Pile Industry. A paper presented before the American Concrete Institute on February 9, 1917, by Charles R. Gow.

wood piling, and the use of iron and screw piles was more or less familiar to the engineers of that period. Sand piles had been used in a limited way, and the writer recalls having seen light steel shells filled with concrete and buried in the ground as foundation supports during the late years of the last century.

"It was generally recognized, however, that wooden piling, although relatively cheap, was poorly suited for many of its requirements. The necessity of cutting wooden piles below permanent ground water level frequently required excessive amounts of excavation and correspondingly large amounts of masonry to bring the footings to grade. If the ground water level was in any way liable to future depression, there was justification for more or less apprehension as to the future integrity of the structure which the piles supported. Again, the amount of loading which could be applied safely on wooden piles was comparatively small . . . . .

"These and other considerations have called for the exercise of ingenuity on the engineer's part in providing a satisfactory substitute for wooden piles which would eliminate their disadvantageous features. The general adoption of concrete as a building material during the late years of the past century naturally led to its introduction for various structural purposes, and it was soon applied as a means to take the place of wooden piling . . . . .

"In the development of the concrete pile industry there have ensu ed two distinct types or groups, viz., those which are cast on the surface and later driven in substantially the same manner as are wooden piles, and the so-called 'built-in-place' piles, which are cast in their final position after the necessary

opening in the soil has been obtained by means of temporary or permanent forms or shells . . . "

Each of these two types of pile has certain disadvantages. The precast pile is usually very expensive, and its use necessitates the provision of a large area of level ground for casting purposes somewhere near the site of the proposed work. Again, although a precast pile can be driven, it is often jetted into the ground, and this operation involves pumping machinery and a large supply of water. It may be noted, however, that precast piles of a maximum length of one hundred and six feet and twenty inches square have been driven successfully,* but the handling of piles of this length is a delicate operation, and their cost is exceedingly high. The greatest disadvantage of the cast-in-place pile is its limited length. The maximum length of this type of pile at the present time is about forty feet, which, of course, makes it useless in many places. The cast-in-place pile, however, has several advantages over the precast pile in that it requires no large casting space, and seldom has to be jetted into the ground. Pile for pile, the cost of a cast-in-place pile, although usually less than that of a precast pile, is considerably greater than that of a wooden pile.

It can be seen, therefore, that the development of a pile which would combine low cost with long lengths and permanence, would aid materially in the solution of many foundation problems. With this object in mind, engineers have developed

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*Pier No. 35 of the San Francisco Harbor Development.

what is known as the "composite pile", which consists essentially of a concrete pile superimposed upon a wooden pile with the joint below permanent ground water level.

Among the first composite piles driven in this country were those placed in 1905 under the station platforms of the Hoboken, New Jersey, terminals of the Delaware, Lackawanna & Western Railroad. These piles were driven in the following manner: The wood piles were driven until the heads of the piles were about a foot above the ground, and then a square box consisting of two-inch planks eight feet long was constructed upon the head of each pile. A wooden follower was inserted through this box and rested upon the head of the wood pile. By means of a collar near the top of the follower, the square box was driven into the ground with the pile. The follower was then withdrawn and the box filled with concrete, thus insuring safety against decay of the wooden pile heads. The permanent ground water level in this instance was about eight feet below the surface of the ground.

From time to time in different parts of the Unites States attempts were made to drive composite piles, but these piles were usually designed to suit the particular problem under consideration, and the results obtained were not satisfactory enough to justify the adoption of this type of pile for quantity production on a commercial basis.

In the early part of 1917, an eminent engineer, in concluding a paper on piling, made the following statement:*"It is

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*<u>History and Present Status of the Concrete Pile Industry</u>. A paper presented before the American Concrete Institute on Feb. 9, 1917, by Charles R. Gow.

probable that the combination wood and concrete pile might more often be used than is customary. Where the conditions impose only a very moderate load upon each pile, and when large numbers of piles are required in the aggregate, a very considerably economy will result if the wooden pile is used in conjunction with a hollow pipe or box follower, so as to permit the head of the wooden pile to be driven below ground water level while the follower portion is filled with concrete."

During the year 1916, the Raymond Concrete Pile Company of New York undertook the development of what is now known as the Raymond Composite Pile. Remarkable success has been obtained with this pile in lengths ranging from forty to one hundred feet, and thousands of them have been driven and are now being driven in different parts of the United States.

III. METHOD OF PLACING THE PILE

The Raymond Composite Pile consists essentially of the upper or heavier section of a Standard Raymond Concrete Pile superimposed upon a wooden pile. The joint between these two piles is carefully constructed and well reinforced, and their relative lengths are such that when the composite pile is completed this joint is always below permanent ground water level. Plate I shows the general arrangement of a Raymond Composite Pile.

When the wooden piles are delivered to the job, they are first unloaded in what is known as the "trimming yard". Here they are carefully selected as to size and straightness, and the accepted piles taken to the heading machine. (Plate 1-A) The principal parts of this machine are a set of rotating knives and a circular saw. The rotating knives trim the head of the wooden pile to a uniform diameter of nine and three-eighths inches over a length of nineteen or twenty inches, and the circular saw cuts off the top of this trimmed section so that a tenon is formed nine and three-eighths inches in diameter and eighteen inches long, the top surface of the tenon being perpendicular to the longitudinal axis of the pile. The headed pile is again returned to the trimming yard, where four three-quarter inch holes twenty inches deep are drilled into the tenon. (See Plate I-B). The pile is then ready for delivery to the pile driver.

Arrived at the pile driver, the wooden pile is hoisted into the leads and a specially constructed steel follower is fitted over the head of the pile. This follower fits snugly

around and over the tenon so as to prevent crushing or "brooming" during driving. After the wooden pile has been driven into the ground to a depth such that the tenon and about a foot of the pile remain above the surface, (Plate I-C), the follower is removed and four five-eighths inch square twisted steel reinforcing bars forty inches long are driven twenty inches into the pile in the four three-quarter inch holes previously drilled into the top of the tenon.

The concrete section of this pile is installed in very much the same manner as the Standard Raymond Concrete Pile. A spirally reinforced steel shell with a plain steel boot at the bottom is placed on the outside of a collapsible steel mandrel or core, which tapers at the rate of .4 of an inch in diameter for each foot of its length. The bottom of this core is so constructed that it fits closely around and over the tenon of the wooden pile and the reinforcing rods extend up into the core. (See Plate II) This combination is then driven farther into the ground until a proper penetration is secured and the top of the wooden pile is known to be below permanent ground water level. The core is collapsed and withdrawn, leaving the wooden pile with its reinforcing bars and the steel shell in the ground. After a careful inspection of the joint and the interior of the shell, a little cement grout is poured around the tenon of the wooden pile and the balance of the shell is filled to cut-off grade with concrete of a one-three-five or a one-two-four mixture.

In this manner a pile is obtained which is practically permanent. Another advantage of this kind of pile is that

it can be driven in great lengths. Wooden piles have been driven as long as eighty feet and then topped off with twenty-two feet of concrete pile, making a total length of a little more than one hundred feet. The cost of composite piles is considerably less than that of all-concrete piles of equal lengths, and the composite pile has all the advantages of an all-concrete pile in the matter of saving in excavation, masonry, pumping, shoring, and other items.

The load allowed on composite piles is usually twenty-five tons per pile. There are many formulae in existence for calculating the bearing power of piles, but the most common of these, and the one most generally used by engineers, is known as the Engineering-News Formula, viz:

$$P = \frac{2\ W\ h}{S + .1} \qquad \text{in which}$$

P = safe load on piles in pounds
W = weight of falling part of hammer in pounds
h = distance of fall of hammer in feet
S = final penetration in inches per blow
Factory of safety = 6

This formula applies only to steam hammers and in the event that a drop hammer is used, the formula is written:

$$P = \frac{2\ W\ h}{s + 1}$$

in which the letters have the same meaning as in the steam hammer formula, and in which the factor of safety is also 6. All Raymond Composite Piles are driven with a steam hammer having a ram weighing five thousand pounds and falling three feet, and driving is continued until the final penetration is one-quarter of an inch per blow. Substituting in the formula:

$$P = \frac{2 \times 5000 \times 3}{.25 + .1} = 85714 \text{ lbs.}$$

This would indicate that a pile driven to the proper penetration would have a carrying capacity of practically forty-three tons, and actual load tests in the field prove that these piles will carry even more than forty-three tons, with only a slight amount of settlement. However, in order to provide an additional safety factor and to preclude the possibility of settlement, the maximum load placed on a composite pile is twenty-five tons.

Since the concrete portion of the Raymond Composite Pile is tapered, there is considerable friction set up between the sides of this concrete pile and the surrounding earth when the load is placed on the pile. The amount of this friction depends entirely on the nature of the soil into which the pile is driven. The friction will be greatest in clay and dry sand and least in soft loam and loose fills. The importance of this feature of the Raymond Composite Pile lies in the fact that an appreciable part of the total load which is placed on the composite pile will be transmitted to the surrounding soil by the tapered concrete section of the pile and the full twenty-five ton load does not often reach the head of the wood pile.

The stresses developed in a pile due to the vertical load are not the only stresses to be taken care of. When a pile is being driven into the ground adjacent to piles already driven, certain indeterminate lateral stresses are set up in the surrounding soil which react on the piles already in the ground. There is also a possibility, especially in newly filled ground, of a lateral movement of the entire mass of

soil under consideration and a consequent tendency to bend over or shear off any piles driven through this soil. These lateral stresses are of some importance, no matter what type of pile is being considered, but they are especially important in the case of a composite pile where the joint is undoubtedly the critical section.

IV. SHOP TESTS

In determining the most desirable type of joint for a composite pile, there were two important factors to be given consideration. First, the joint must be strong enough to resist the stresses, vertical and lateral, which act upon it; and second, the joint must be so constructed that its use would not involve too many complicated operations in the field.

In order to determine the relative strength of various types of composite pile joints, some actual load tests were made on different kinds of composite piles at the shops of the Raymond Concrete Pile Company at Harvey, Illinois. Three different types of joints were designed, and several full size composite piles of each type were made. These piles were loaded and tested in various ways, and the general arrangement of the tests together with the tabulated results are shown on Plates III to XXV, inclusive.

Plates III, IV, and V show the methods used and results obtained in testing a composite pile as a simple beam over a span of forty-four feet. As was expected, these piles showed rather large deflections under relatively small loads. The most important feature of these tests, however, is the fact that in two out of three tests (A2 and A3), the wood pile broke while the joint remained uninjured, and in the other test (A1) the failure started in the wood pile, which split lengthwise, although the ultimate failure was in the joint. The conditions of this test could hardly obtain in a pile driven into the ground; however, the test indicates that for a full length pile, loaded as shown in Plate III, the critical section is not the joint, but is that

part of the wooden pile where the maximum stress exists.

Plate VI shows the details of the joint used for tests A, B, C, and F.

Plates VII, VIII, and IX indicate the method and results obtained in testing a composite pile as an unsupported column forty-five feet long. The pile was tested in a horizontal position and the deflection due to the weight of the pile itself was taken up by a counterweight, as indicated in the lower left-hand corner of Plate VII. The load was applied by means of an hydraulic press which registered in tons. It should be noted in connection with this test that the deflection was all in the wood pile until a load of twenty-eight tons was reached. At this point, the joint began to open, and continued to open more as more load was applied. At twenty-five tons, which is the load usually placed on a composite pile, the deflection was one and one-sixteenth inches, and there was no indication whatever of strain in the joint. It must also be remembered that a pile driven into the ground would not act as an unsupported column, but would receive more or less lateral support from the surrounding soil, the amount of this support depending upon the nature of the soil, and that this test, therefore, imposed upon the pile conditions far more severe than those existing in the field.

Plates X, XI, and XII show the results obtained by testing a composite pile as a cantilever with the maximum bending moment at the joint. The purpose of these tests was to determine the lateral strength of the joint in the event that the wood pile were driven into a solid subsoil which would hold it

firmly while the concrete portion of the pile remained in a loose top soil, which would at some time be capable of lateral movement. The average breaking load was about three tons, and the tests indicate very little except that the joint would have very little lateral strength if there were a clearly defined fault in the soil at or near the joint.

The "D" tests, (Plates XIII, XIV, and XV), were made on a type of joint which differed from the joints of the preceding tests and the piles were tested as simple beams over an eleven-foot span. This type of joint proved to be very rigid, and the deflections recorded were relatively small. The breaking loads, however, were also small, and about all that can be said concerning this type of joint is that although it is a fairly rigid joint, it is too weak a joint to receive serious consideration.

The "E" tests (Plates XVI, XVII, and XVIII) were made with a third type of joint, and in this case the piles were again tested as simple beams over an eleven foot span. This type of joint tested very well, and test E1 developed the maximum bending moment (see Plate XXV) among all of the sixteen tests. The joint proved to be quite rigid, it sustained fairly heavy loads, and was the type of joint ultimately adopted for the Raymond Composite Pile.

In the last series of tests (Plates XIX, XX, and XXI) the piles were tested as simple beams over a span of eleven feet with the same type of joint used in the first three series of tests. These piles also tested very well, sustaining comparatively heavy loads, but with an average deflection somewhat greater than that of the "E" tests. This type of joint, however, is more complicated than the "E" joint, and its use

would involve considerable time and labor in the field.

Plates XXII, XXIII, and XXIV are a series of load deflection curves for the "D", "E", and "F" tests. These tests were made with the same conditions of loading and the same span lengths, and a study of these curves will give some idea of the relative merits of these three types of joints.

Plate XXII shows the load-deflection curves for the "D" tests. This type of joint proved to be fairly rigid, but the breaking loads were low, the average breaking load being about three tons. For light loads, this joint was as rigid as the "E" joint, and considerably more rigid than the "F" joint, but, because of its low breaking strength, it was not given further consideration.

The load-deflection curves for the "E" tests are shown on Plate XXIII. Examination of these curves brings out several points. First, this type of joint is as rigid as the "D" joint, but has a much greater breaking strength, and its breaking strength is equal to that of the "F" joint, but it has greater rigidity. In other words, the "E" joint combines the rigidity of the "D" joint with the strength of the "F" joint, without the weakness of either of these two joints; and the "E" joint is therefore better than either of the other two. A second point that should be noted in connection with these curves is that the three curves stay quite close together over their entire lengths. This fact proves that the three piles tested were approximately equal in strength, and it may be taken to indicate that if a number of piles of this type were installed in a foundation, they too would be of uniform strength. It may also be noted that among the nine piles

tested in the D, E, and F tests, pile E-1 supported the greatest load.

Plate XXIV shows the load-deflection curves for the "F" tests. Test F-1 produced a very uniform curve, and the load sustained was quite large. However, the deflection was considerably greater than that of any other test. Test F-2 worked out very well, both as to load and deflection. The curve for test F-3 is quite irregular and seems to indicate that some of the parts were slipping as the load was increased, although during the test nothing could be observed which would prove this. There is a decided "break" in the curve between one thousand pounds and twenty-five-hundred pounds, another "break" between sixty-five hundred pounds and seventy-five hundred pounds, and at eleven thousand five-hundred pounds the slipping continued rapidly until the ultimate was reached. Unlike the "E" curves, these "F" curves did not stay close together, and the specimens tested were not uniform in strength. This fact may be taken to indicate that the strength of a joint as complicated as this one depends a great deal on the workmanship; and the possibility of installing in a foundation a number of piles of uniform strength with this type of joint would be rather remote. It should also be noted that the initial (no-load) deflection of these "F" tests was just double that of the "E" tests, indicating that the "F" joint is less rigid than the "E" joint. Another disadvantage of this "F" joint is that its use would involve considerably more of labor and material than the "E" joint, and since it could not offset this disadvantage by showing greater strength or greater rigidity than the "E" joint, it was not adopted.

Unfortunately, the joint used in the "E" tests was not tested as an unsupported column (similar to the "B" test). However, since the strength of a column depends to a large extent upon its rigidity, and since the "E" joint proved itself more rigid than the "pin" type of joint, it is safe to say that in a column test similar to the "B" test, the "E" joint would have given just as good if not better results than were obtained in the "B" test.

Plate XXV is a tabulation of the bending moments and stresses that existed in the various specimens at the ultimate load. The methods of calculation and formulae used in computing these stresses are shown on the two pages immediately following Plate XXV. Because of the complicated nature of all these joints it is practically impossible to calculate accurately the actual stresses which existed in the several members of the joint at the ultimate load. In calculating the stresses shown on Plate XXV, it was assumed in each case that the member under consideration was resisting the entire bending moment. As can readily be seen, these stresses are therefore somewhat higher than those which actually existed. However, in this particular case the actual stresses are not of paramount importance. The principal object of all of these tests was to obtain a comparison of the several types of joints, and, since all of the results set down on Plate XXV were calculated on the same basis of reasoning, these results may properly be compared.

An examination of the stresses developed in the "A" tests indicates that of the three materials (wood, steel, and concrete) used in the joint, the wood was the most dangerously

stressed. The fibre stresses in Column 3 and the shearing stresses in Column 9 are all above the ultimate for short-leaf pine. The fibre stress in the reinforcing bars (Column 5) is also very close to the ultimate, but the fibre stresses in the pins (Column 8) are but little above the allowable safe stress. The concrete fibre stresses (Column 4), while somewhat above the safe stress for reinforced concrete, are still only about one-half of the ultimate. The results of these "A" tests indicate that the several members of this "pin" type of joint act together fairly well, but it must be remembered that in two of these three tests the failure occurred not in the joint, but in the wood pile, and in the third test the failure started in the wood pile, although the ultimate failure was in the joint.

Although the specimen in the "B" test was not actually broken, the stresses calculated are quite high. The stresses in the wood (Columns 3 and 9) are well above the ultimate. The fibre stress in the reinforcing bars (Column 5) is close to the ultimate, although the fibre stress in the pins (Column 8) is only a little above the safe working stress. The concrete stress (Column 4) is again somewhat higher than is usually allowed, although it is only about one-half of the ultimate.

In all of the specimens tested in the "C" tests, the failure occurred in the wooden portion of the joint. Examination of the stress sheet shows very clearly the reason

for this. The fibre stresses (Column 3) and the shearing stresses (Column 9) are all far above the ultimate. The fibre stresses in the bars (Column 5) are slightly above the ultimate but the concrete stresses (Column 4) and the pin stresses (Column 8) are well under their respective ultimate strengths.

The critical section in the "D" tests was the concrete tenon imbedded in the head of the wood pile. All of the three specimens tested failed because of the crushing of the tenon. The fibre stresses (Column 4) developed in this tenon are just about twice the ultimate. The fibre stresses in the wood (Column 2) are only a little above the allowable working stress, although the stresses in the bars (Column 5) are very close to the ultimate. The bond stresses which apparently existed between the bars and the wood are shown in Column 10. It must be remembered in this connection that these were square twisted bars, and that they were driven into the wood in holes whose diameter was somewhat less than the diameter of the bars. The screw action of these bars would undoubtedly develop considerable bonding strength between the bar and the wood, but it hardly seems reasonable to expect that enough strength could be developed in this manner to resist the stresses shown in Column 10.

In all of the "E" tests, the failure occurred in the concrete. The stresses which apparently existed in the concrete (Column 4), while somewhat greater than the allowable unit stress, are yet less than the ultimate. The stresses in the wood (Column 2) are very close to the ultimate,

but the steel stresses, (Column 5), and the bond stresses, (Column 10), are far higher than could possibly have existed. It is these steel and bond stresses which help to explain the reasons for the failure in the concrete. In order to develop any tensile stresses in these reinforcing bars, it would be necessary for the bars to be firmly held at both ends. The amount of stress that could then be developed would depend upon how firmly the bars were held. Since the stresses shown in Columns 5 and 10 are far in excess of any stresses which could have existed, it is safe to conclude that the reinforcing bars were not firmly held at both ends, but that they were slipping out of the wood pile. This conclusion is justified by the fact that an examination of the specimens after they were broken indicates clearly that the rods were slipping out of the wood pile. In addition to this, the "breaks" in the load-deflection curves on Plate XXIII show that some part of the joint was slipping as the load was applied, and in the light of additional evidence it may safely be concluded that these "breaks". indicate the slipping of the reinforcing bars out of the wood pile. When the initial load was applied to the specimen, the top of the wooden tenon was firmly pressed against the concrete, and the bending moment was resisted by a solid circle of concrete approximately 13.9 inches in diameter. As the load was increased and the reinforcing bars started to slip out of the wood pile, the concrete moved away from the top of the wooden tenon, and there was a small space between these two materials. The bending moment was then resisted by a hollow circle of concrete whose outside diameter was approximately 13.9 inches,and

whose inside diameter was 9.5 inches. This hollow circle or ring of concrete was not strong enough to resist the bending moment, and it was thus that the failure occurred. (See pictures of "E" tests on Plate XVIII.) The fact that this type of joint failed in the manner described above does not indicate that a composite pile with this type of joint would have the same weakness under actual working conditions. When the pile is driven into the ground, the heavy vertical load which is placed upon it will keep the concrete and the wood pressed tightly together, no matter how great is the lateral thrust.

In the "F" tests, all of the failures occurred in the wood. The wood tenons broke off in all of the three tests, and in two tests the steel pins sheared the wood noticeably. The fact that these pins were shearing the wood is apparent in the load-deflection curves on Plate XXIV. The deflections plotted are not uniform. There are a number of little "breaks" in these curves which in all probability indicate the shearing of the wood by the steel pins. The fibre stresses in the wood (Column 3) and the shearing stresses (Column 9) are all far above the ultimate. The concrete stresses (Column 4), while not quite up to the ultimate, are considerably above the safe working stress. The stresses in the bars (Column 5) are above the ultimate, but the pin stresses (Column 8) are well within the elastic limit.

In summing up these tests, it may be stated that they brought out some very interesting facts as to the strength of these piles and the relative merits of the different types of joints. The type of joint used in the "E" tests proved itself

the best. It was the stiffest joint, so far as deflection is concerned; it supported the greatest load; it showed up well on the stress sheet, and its construction involves less time, labor, and material than the other joints. It was this "E" joint that was therefore adopted, although some slight modifications of its construction have recently been made.

V. FIELD TESTS

The proper method of determining the safe bearing power of a pile by means of a formula, is a subject which has brought forth considerable discussion among engineers. The determination of the bearing power of a columnar pile, i.e. one which is driven through semi-fluid material and has its lower end resting on a hard stratum, is simple enough. Such a pile is designed as a column using the ordinary column formulae, although the unsupported length used in the formula may be somewhat less than the length of the pile if the surrounding soil is capable of giving the pile lateral support.

For a friction pile, i.e. one which supports its load by means of the friction developed between the sides of the pile and the surrounding soil, there are a number of formulae. These formulae take into consideration a variety of conditions which may exist during the driving of a pile. One of the simplest of these formulae and the one must generally used by engineers is known as the Engineering-News formula, and for drop hammers this formula is written as follows:

$$U = \frac{12\ W\ h}{S + 1}, \quad \text{in which}$$

U = ultimate load on pile in pounds
W = weight of hammer in pounds
h = fall of hammer in feet
S = final penetration of pile in inches
Safety factor = 6

Several other interesting but more complicated formulae for drop hammers are the Redten-Bacher and the Rankine formulae, written as follows:

$$\text{Redten-Bacher} \ldots \quad U = -\frac{E\,s}{L} + \sqrt{\frac{2\,E\,w^2h}{aL\,(w+p)} + \left[\frac{E\,s}{L}\right]^2}$$

$$\text{Rankine} \ldots \quad U = 2\left\{-\frac{E\,a\,s}{L} + \sqrt{\left[\frac{E\,a\,s}{L}\right]^2 + \frac{E\,a\,w\,h}{L}}\right\}$$

In the Redten-Bacher formula the safety factor is six, in the Rankine formula the safety factor is ten, and in both formulae the letters have the same meanings, as follows:

U = ultimate load on pile in pounds
E = modulus of elasticity of pile
s = final penetration of pile in feet
p = weight of pile in pounds
L = length of pile in feet
a = cross-sectional area of pile in square feet
w = weight of hammer in pounds
h = fall of hammer in feet.

These formulae are not in general use, but they give some idea of the number of factors which enter into the determination of the safe bearing power of a friction pile.

It has often been noted that the apparent ultimate load for a pile as obtained from a formula does not check up with the load observed when the pile is actually tested. In the case of the more simple formulae, this discrepancy may be due to the fact that several important factors are omitted from the formula. In the case of the more complicated formulae it is difficult to imagine that anything has been omitted unless the experimental data on which the formulae are based do not cover the wide variety of conditions which actually

exist in the field. In most of these formulae an attempt has been made to make the formula fit all conditions by the use of a sliding safety factor, i.e. to use a different safety factor for each different driving condition. This, however, does not give absolute satisfaction.

In this connection there are several conditions well known to engineers engaged in pile driving work which may help to explain the apparent weaknesses of the formulae. In the case of a pier where a group of twenty-five or thirty piles are driven fairly close together, the calculated safe load of the first pile driven may be relatively small. If, after all the other piles in the pier are driven, this first pile is loaded and tested, it will often be found to sustain considerably more than the calculated load. The reason for this is obvious, since the displacement and compression of the soil brought about by the driving of succeeding piles would cause the soil to pack more tightly around the first pile, thereby giving it greater bearing power. It can readily be seen that this condition can hardly be expressed in a formula, since it depends on the number of piles in the pier, the arrangement and spacing of the piles, and on the nature of the soil. The determination of constants to cover this condition would require a great deal of experiment; the number of constants required would be large, because of the wide variety of soils; and even if a series of constants were determined their practical application would present a difficult problem.

There is another condition often encountered in pile driving operations which affects not only groups of piles but

even isolated piles with no other piles anywhere near them. This condition is often referred to as the "come-back" of the soil. When a pile is driven into the ground a distance of thirty or thirty-five feet and then allowed to rest for ten or twelve hours, it will often be found that several blows of the pile hammer are required to start the pile moving again, and even after it is moving, the penetration per hammer-blow will usually be somewhat less than the final penetration per hammer-blow of the original driving. The probable explanation of this is that during the driving the energy produced in the pile by the repeated blows of the hammer on the top of the pile agitates the little particles of soil along the sides of the pile so that they are not able to grip the sides of the pile, and resist its further penetration. As soon as the driving stops, these soil particles come to rest, and the accumulated pressure in the soil caused by the displacement of the pile tends to readjust and equalize itself. This readjustment of soil pressures forces the soil particles nearest the pile into close contact with the sides of the pile, and the friction between the pile and the soil is increased. This internal action in the soil depends entirely upon the nature of the soil itself, and it is evident that this is another factor which cannot very well be incorporated into a formula, and that any bearing load calculated on the basis of the penetration per hammer-blow during the original driving would not be correct.

It can readily be seen, therefore, that the apparent

safe bearing power of a pile as calculated by a formula cannot be absolutely relied upon. The best method of determining just what load a pile will safely carry is to place an actual test load on the pile. This does not mean that every pile driven on a particular job must be loaded and tested. In an ordinary building foundation a load test on several piles at opposite ends of the site would be sufficient. The safe load for the pipes tested will, of course, be known, and a comparison of their driving records with the driving records of the other piles driven will give an accurate idea of the bearing power of all of the piles.

Plate XXVI gives an idea of the apparatus used for testing a pile. The two upper sketches show a plan and a side elevation of the testing platform with a sand box, and the lower sketch shows the general arrangement of the testing apparatus when the pile cut-off is below ground level. Whether the pile to be tested is wood, precast, concrete, cast-in-place concrete, or composite, the procedure is practically the same. After the pile has been driven to the desired penetration, the top of the pile is cut off perfectly level and a 5/8" or 3/4" bolt 14" or 15" long is placed in a vertical position firmly in the head of the pile. A convenient benchmark is then selected and an elevation taken on the top of the bolt and referred to the bench mark. A round iron plate approximately 20" in diameter and 2" thick with a small hole in the center is then slipped down over the bolt and placed on the top of the pile. This plate must be absolutely level in all directions. Two 18" I-beams are placed parallel to one another on

top of this plate, one I-beam each side of the bolt, and these I-beams are connected at several points by diaphragms which act as stiffeners to prevent their turning over. Six 12" x 12" timbers, each 16 ft. long, are placed on 2'6" centers across the top of the I-beams and a 2" plank floor is spiked on top of the timbers. This entire structure must be carefully balanced in all directions, using the pile as a center, and a small hole is left in the plank flooring so the level rod can be placed on the top of the bolt. A small timber crib is then built under each of the four corners of the platform, and the ends of two of the 12"x12" timbers are supported on these cribs with wooden wedges. The purpose of this is to prevent the platform from tipping off of the pile while the load is being placed, the wedges being easily removable when it is desired to allow the full load to rest on the pile.

Various materials are used for loading test piles, such as sand, gravel, cement in bags, steel rails, reinforcing steel, pig iron, heavy iron castings, and water. The use of water necessitates a water-tight tank; cement must be protected from the weather, steel and iron are not often available, so that sand or gravel are most often used, and sand is preferable, as it weighs more per cubic foot than gravel. When sand is used, a box is built on top of the platform, and a pipe six or more inches in diameter is placed in the center of the box directly over the pile so that the level rod can be inserted through the pipe and placed on top of the bolt in the pile head. The dead load of the platform and box is calculated, and enough sand is distributed evenly over the bottom of the box to bring the load to five tons. The wedges are then removed

from the four corners of the platform and the load allowed to rest on the pile for several hours, level readings being taken at intervals. The loading is continued in five-ton increments, and each new load is allowed to rest on the pile for several hours while level readings are taken. Whenever a settlement is noted, no more load is added till the pile stops settling. In this manner, the load is brought up to the desired total, and the pile is allowed to carry the total load until no additional settlement is noted over a twenty-four hour period. The load is then removed and the box and platform taken down and a final level reading is taken on the pile to determine whether or not it has recovered some of its settlement.

All Raymond Composite Piles are driven with a No. 1 steam hammer having a ram weighing five thousand pounds and falling three feet. These piles are driven to a final penetration of one-quarter of an inch per hammer blow, and by substituting these quantities in the Engineering-News formula, it would appear that a pile so driven would safely support a load of 43 tons. As has been shown, the calculated safe bearing power of a pile, as determined from a formula, does not always check with the actual safe bearing power as determined by a load test, and accordingly a number of Raymond Composite Piles driven in different kinds of soils in different parts of the country have been loaded and tested.

Plate XXVII is a photograph of a test on a Raymond Composite Pile driven in the soft mud of the New Jersey Meadows at Warners, New Jersey. In this case the wooden section of the pile was 20 feet long, and the concrete section was 13 feet long. The load was applied in 10-ton increments, and no

settlement was recorded when 40 tons had been reached. At 50 tons a settlement of .125 inches or 1/8 of an inch was noted, and at 60 tons the total settlement was .187 inches, or 3/16". The designed load in this work was 25 tons per pile.

Plate XXVIII is a photograph of a load test on a Raymond Composite pile at the Army Supply Base at New Orleans, Louisiana. These piles were driven in the soft alluvial mud which is found along the banks of the lower Mississippi River. The wooden section of this test pile was 50 feet long, and the concrete section 10½ feet long. In this test no settlement was noted up to 40 tons, and since this was more than twice as much load as the pile was expected to support, the loading was not carried any further. The designed load in this case was 16 tons per pile.

VI. RECENT DEVELOPMENTS

A great deal of thought and study has been given toward the improvement of the Raymond Composite Pile, and considerable of time and money have been expended in working out and testing these various new developments. The first changes that were made were directed toward the simplifying of the actual field operations. These changes were chiefly in the design and construction of the cores, and their object was to improve the method of handling and placing the reinforcing bars.

When the first Raymond Composite Piles were driven, the four dowel rods were not placed in the head of the wood pile while it still projected above the ground. Instead, the wood pile was followed down to its proper grade by the steel core, and then the four dowel rods were placed just before the shell was filled with concrete. In order to place these dowel rods in the head of the wood pile which was then 10 or 15 feet below the surface of the ground, a piece of ordinary cast iron pipe about 2 inches in diameter and 10 or 15 feet long was placed as nearly as possible over one of the four holes previously drilled into the head of the wood pile. The dowel rod was then dropped through the pipe into the hole, and was driven down by means of a small weight which slid up and down in the pipe either on a cord or a thin, light rod. As can readily be seen, this method of placing the reinforcing rods was not entirely satisfactory, and if the concrete section of the pile were long, thereby necessitating a long piece of pipe, the operation would keep two men busy most of the time.

The first improvement over this method of placing the reinforcing bars is shown on Plate II. This drawing shows the interior arrangement of the lower portion of the pile core in position over the head of the wood pile and ready to follow the wood pile into the ground. The reinforcing rods are in place in the head of the wood pile, and they project up into the interior of the core. The placing of these rods is a simple matter, inasmuch as the head of the wood pile is above the surface of the ground, and is easily accessible, and all that is necessary to do is to drive the four rods into the four holes previously drilled.

Several additional improvements have been made on these cores which do not appear on Plate II. In some instances it was found that in using the arrangement shown on Plate II the reinforcing rods would somehow become bent during the driving of the wood pile, and when the core was withdrawn the bent rod would be brought up with the core. This not only necessitated the old method of dropping a rod through a long pile to replace the bent rod, but very often the bent rod itself caused considerable trouble before it could be extracted from the core. To overcome this difficulty four piles were welded onto the center plunger of the core, and as the core was lowered over the top of the wood pile the reinforcing rods entered these pipes, and were prevented by the pipes from becoming entangled in any of the interior mechanism of the core. Recently several cores have been built in which these pipes were made an actual part of the center plunger casting. This forms a very rigid guide for the rods, and works out very well.

The last change or improvement in the construction of the cores is the addition of a small collar which is fastened firmly to the center plunger just at the top of the reinforcing rods. The purpose of this collar is to do the actual work of driving the reinforcing rods. All that is done by hand is to stand the four rods up in the holes already drilled, and as the core is lowered onto the wood pile the four rods enter the four pipes inside the core, are caught by this collar on the center plunger, and are forced down into the wood pile by the weight of the core.

The part of the composite pile that next received attention was the "pile boot". This boot is pressed from No. 18 gauge sheet steel, and it is used at the bottom of the concrete section of the pile. The position and relative size of this boot can be seen on Plate I-C. The original form in which this boot was made is shown in detail on Plate XXIX. This type of boot worked out very well in most cases. However, it was found that in very wet soils, and especially in highly saturated sand and soft mud there was a tendency for the water to work its way into the boot, bringing with it fine particles of mud and sand, thereby impairing the strength of the concrete in the bottom of the boot. This mud and water seemed to come in from under the boot, and apparently worked its way between the underside of the boot and the shoulder of the wood pile. In order to eliminate this difficulty, the method of making the boot was changed. Instead of turning under the lower end of the boot and allowing it to rest on the shoulder of the wood pile, the lower end of the new boot came down alongside of the wood pile, and was fastened to the body of the wood pile with heavy nails. Plate XXX

shows this new arrangement for the pile boot. In addition to having the boot extend down over the outside of the wood pile and having it spiked to the wood pile, this new scheme involved the use of a laminated wood ring or a pressed steel ring on the shoulder of the wood pile. Plate XXX shows one of these wooden rings in place. The purpose of these wood or metal rings is to seal up the connection between the wooden and the concrete pile so as to insure as nearly a perfect joint as possible.

Considerable study has been given recently toward improving the method of fastening the concrete and the wooden piles together. As can be seen from the E tests, the four twisted dowel bars add considerable strength to the joint for the resisting of lateral stresses, and they undoubtedly develop a relatively high bond stress with the wood. There is, however, a condition which can exist in the field which this method of reinforcing may not be able to take care of. This condition is that of a direct vertical uplift on the concrete portion of the pile. On rare occasions the structure that is to be placed on the piles will be so designed that under certain loading conditions there will be an uplifting force at the top of the pile. This condition is uncommon, and should it be desired to use composite piles under a structure so designed, it would be necessary to design a special composite pile to meet the requirements.

A condition not quite so rare, however, is the heaving action of certain kinds of soils. In a highly elastic incompressible soil, such as certain kinds of wet clay, the driving of piles adjacent to one already driven will cause certain

forces to act on the pile previously driven. These forces will be partly horizontal and partly vertical, and although a measure of their intensity would be hard to obtain, there is no doubt that they exist, and that they should be given consideration.

As can readily be seen, the ability of the composite pile to resist heaving action in the soil when the E type of joint is used depends entirely on friction. This friction exists between the concrete and the outside surface of the wood pile tenon, and between the wood of the tenon and the twisted bars driven into it. The use of the new style boot (Plate XXX) assists greatly in holding the two piles together, but it is thought that some more permanent connection should be designed in the form of a positive lock.

A number of experiments were made with lag screws. The intention was to follow the wood pile down to its proper elevation in the ground without putting any reinforcing material in the joint, and then, after the core had been withdrawn, and just before the concrete was placed, to take a lag screw onto which a long reinforcing bar had been welded and screw it into the top of the tenon. With the lag screw set firmly into the wood and the long rod bonded with the concrete and the rod and screw welded together, it was thought that a very rigid connection would be made. Experimental results, using several different kinds and sizes of lag screws, were disappointing. The lag screws placed with the grain of the wood in this manner did not develop much strength, even when they were screwed in a distance of 16 or 18 inches. In addition to this weakness, considerable difficulty was experienced in screwing the lag screws

into the wood when the head of the wood pile was from 10 to 15 feet below ground. The most interesting thing about these experiments was in connection with the weld between the lag screw and the reinforcing rod. These were simple butt welds made with an electric welding machine, and in each case the welded parts held together in direct tension, while the lag screws were being pulled bodily out of the wood with 3000 and 4000-pound loads.

Another type of lock which was tried out was known as the "bayonet lock". This lock consisted essentially in a small steel cylinder about 2¼ inches in diameter and 7 inches long which was placed vertically in the top of the wood pile tenon. A 7/8" diameter steel pin was placed through the tenon and through the lower end of the steel cylinder to hold the cylinder firmly in the tenon. A 1" diameter hole about 5" deep was drilled down from the top of the steel cylinder, and a vertical slot 3/4" wide and 1/2" deep was cut along one side of this drilled hole. A small pocket was cut out of the cylinder at the bottom of the drilled hole, so that a long bar with about 1½ inches of its lower end bent back at 180 degrees could be inserted into the cylinder with the bent-up part passing down the slot and then rotated about a quarter of a turn so that the bent-up part of the bar would fit into this pocket. This was expected to provide a positive lock and a few of these small steel cylinders were made up and tried out.

The method of installing this type of lock in the field was to place the steel cylinder in the head of the wood pile just before it was followed down into the ground by the concrete pile. The concrete portion of the pile was then driven, and before the shell was filled with concrete

the long reinforcing rod with its lower end bent up was inserted into the cylinder and rotated so that it could not come out.

A number of experiments were made with this "bayonet lock", both in the shop and in the field, but they were all unsatisfactory. The dimensions of the various parts were changed, and the little pocket at the lower end of the cylinder was enlarged so that the rod could be given about a half turn instead of a quarter turn. The vertical slot in the inside of the cylinder was changed to a spiral slot, but not any of these changes gave the desired results, and the experiments were discontinued.

Recently a new type of lock has been developed, the general arrangement of which is shown on Plates XXXI and XXXII. This lock is similar to the one just described above, except that a special thread is cut on the inside of the steel cylinder and a small steel screw is welded onto the end of a reinforcing rod, and is screwed into the cylinder. The operations involved in installing this lock in the field are as follows: The hole for the cylinder or socket and the hole for the 7/8" diameter steel pin are drilled into the wood pile tenon at the trimming yard, using a specially constructed jig as a guide. The wood pile is then driven down to ground level and the socket or cylinder is placed in the top of the tenon and the pin inserted. After the wood pile has been followed down to its proper elevation, the reinforcing rod with the screw welded onto it is screwed into the socket and the pile is ready for the concrete.

A number of experiments have been made in the field with this type of lock, and all of them were successful. One or two shop tests have also been made, and the results obtained are equally encouraging. In cross-bending, this joint proved itself as strong as the "E" joint and somewhat more rigid. In direct tension, the total load sustained was 22,000 pounds, with a total deflection of 3/8". In all of the shop tests the 7/8" diameter pin bent and sheared the wood, although in the cross-bending tests the ultimate failure was in the concrete at the top of the tenon. Several more shop tests are yet to be made, especially a compression test on a full length pile as an unsupported column.

All of the experiments made to date with this type of joint have been very encouraging, and although it has not yet supplanted the "E" joint as a standard for composite piles, it is highly probable that after a little further experiment and some slight modifications, it will supplant the "E" joint and become the standard joint for Raymond Composite piles.

VII. SUMMARY AND CONCLUSIONS

In summing up this paper, it may be stated that the Raymond Composite Pile has helped materially in solving some of the problems of foundation construction. The results obtained under actual working conditions and the results of the shop tests have conclusively proved the strength of the pile. The composite pile is as permanent as an all-concrete pile, since the wooden portion of it is driven below permanent ground water level, and yet it is considerably cheaper than an all-concrete pile of the same length. In addition to its low cost, the composite pile has all the advantages of an all-concrete pile in the saving of excavation, masonry, pumping, and shoring; and it can be driven in almost any desired length.

Although the idea of a combination wood and concrete pile is not a particularly new one, since such piles are known to have been used as early as 1905, the Raymond type of composite pile is new, since it has been in use at this time but about five years. From time to time minor defects have appeared both in the construction of the pile and the method of installing it, but as has been pointed out, changes and improvements have been made to overcome these difficulties. The future will probably bring more changes and improvements, but the Raymond Composite Pile has undoubtedly long since passed the experimental stage.

Up to the present time approximately fifty thousand of these piles have been driven in different parts of the United States in various kinds of soils. The pile is particularly well adapted to foundations which require large numbers of piles, and there can be no doubt but that the Raymond Composite Pile is now prepared to take its place among the standard types of foundation construction.

VIII. THE AUTHOR'S CONNECTION WITH THE RAYMOND COMPOSITE PILE

The author's first personal experience with the Raymond Composite Pile was in the fall of 1919, when he was in charge, for a time, of some construction work where composite piles were being placed.

In the fall of 1920 the author was ordered to investigate and report on a foundation containing approximately 3000 composite piles, in which several defects were apparent. The investigation required about a week's time, and the facts brought to light in this investigation caused some of the changes to be made which are described in Chapter VI of this thesis. The author made the drawings for some of these changes, and conducted experiments and tests in connection with them at the Raymond Concrete Pile Company's shops at Harvey, Illinois. These tests involved the changes in the pile boots, the use of wood rings on the shoulder of the wooden pile, and the fastening of the concrete and wooden piles together.

The author did not make any of the tests described in Chapter IV of this thesis, but simply worked the field notes and data into the form in which they are presented.

This thesis is the first connected account of the history of the Raymond Composite Pile, and the author is indebted to the Raymond Concrete Pile Company for the use of its files and records for the preparation of it.

IX. THE AUTHOR'S PROFESSIONAL EXPERIENCE

August 1915 to February 1916 -- Rodman with New York, New Haven and Hartford Railroad.

March 1916 to July 1916 -- Timekeeper, Raymond Concrete Pile Company.

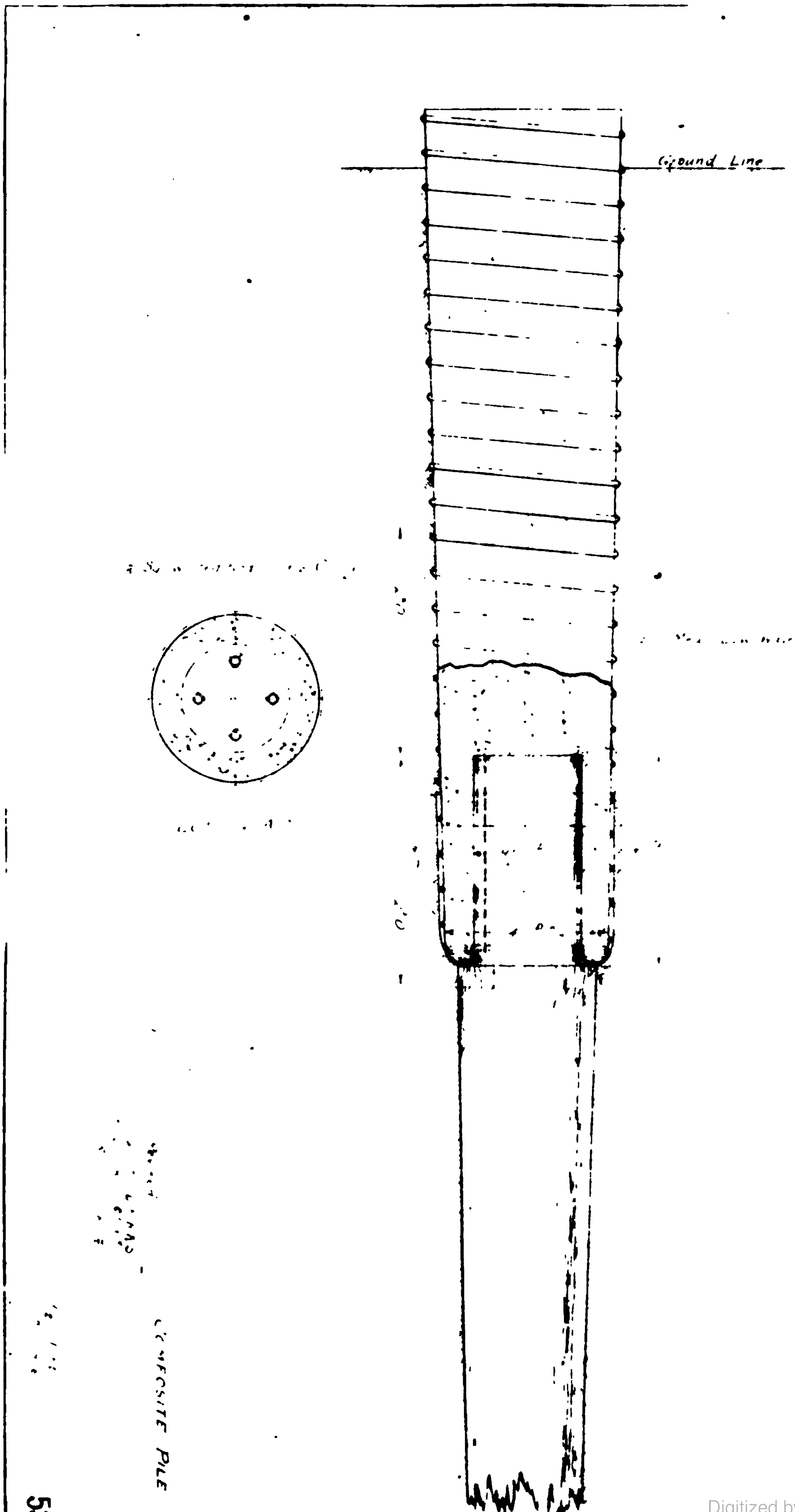
August 1916 to December 1916 -- Field Engineer, Raymond Concrete Pile Company. (Lines and grades on construction work; cost records and progress charts).

January 1917 to April 1920 -- Superintendent of Construction, Raymond Concrete Pile Co. (Docks, piers, bulkheads, shipways, and heavy foundations at Buffalo, New York; Baltimore, Maryland; Norfolk, Virginia; Hog Island, Pennsylvania; Boston, Massachusetts; and Chicago, Illinois).

May 1920 to present time -- Office Engineer and District Superintendent, Raymond Concrete Pile Company. (General office, engineering, and sales work. General supervision of construction work in the middle west, with headquarters at Chicago).

XI. APPENDIX

PLATE I.



COMPOSITE PILE

PLATE I-A

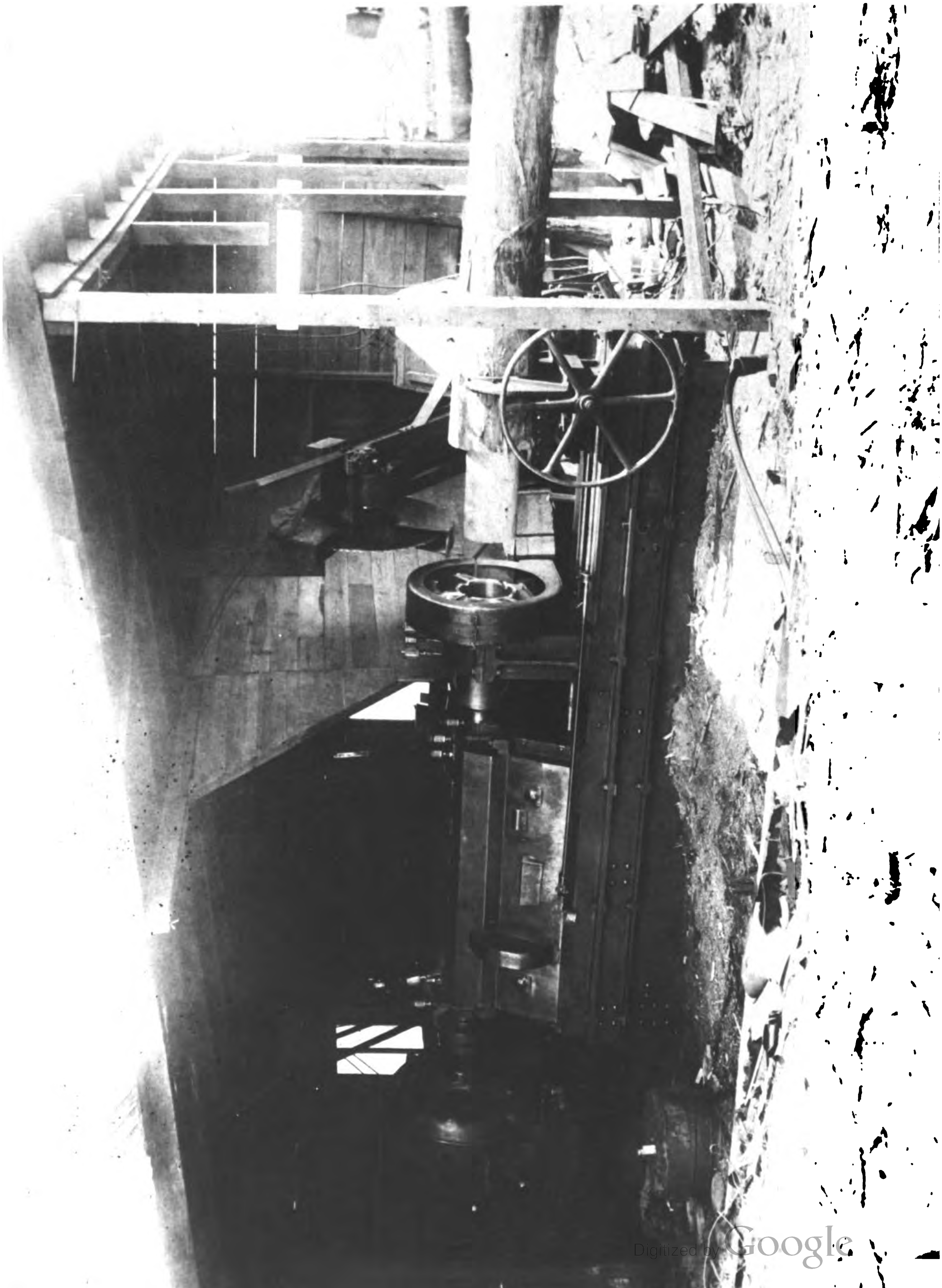


PLATE I-B

PLATE I-C

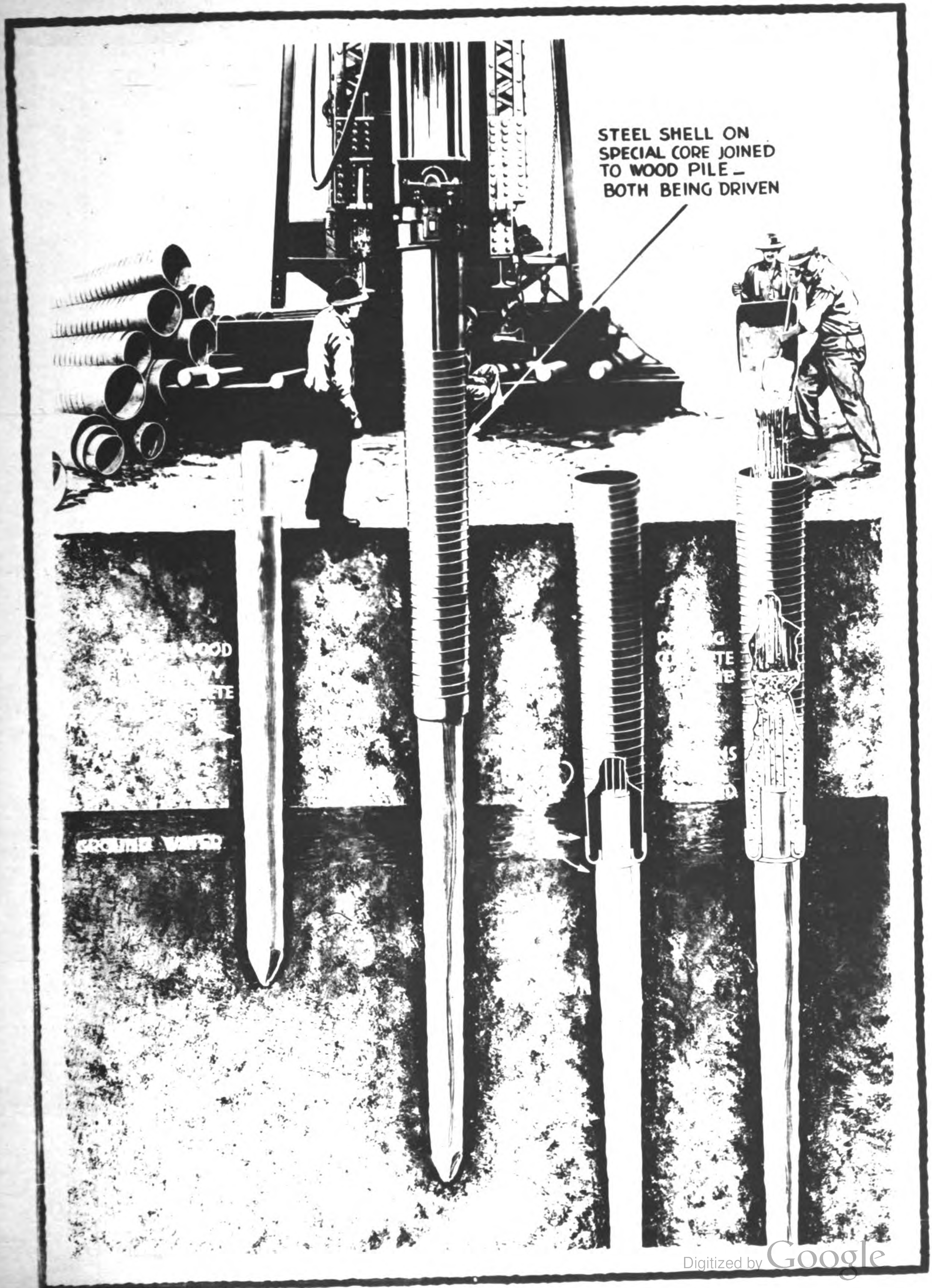

STEEL SHELL ON
SPECIAL CORE JOINED
TO WOOD PILE—
BOTH BEING DRIVEN

PLATE I-C

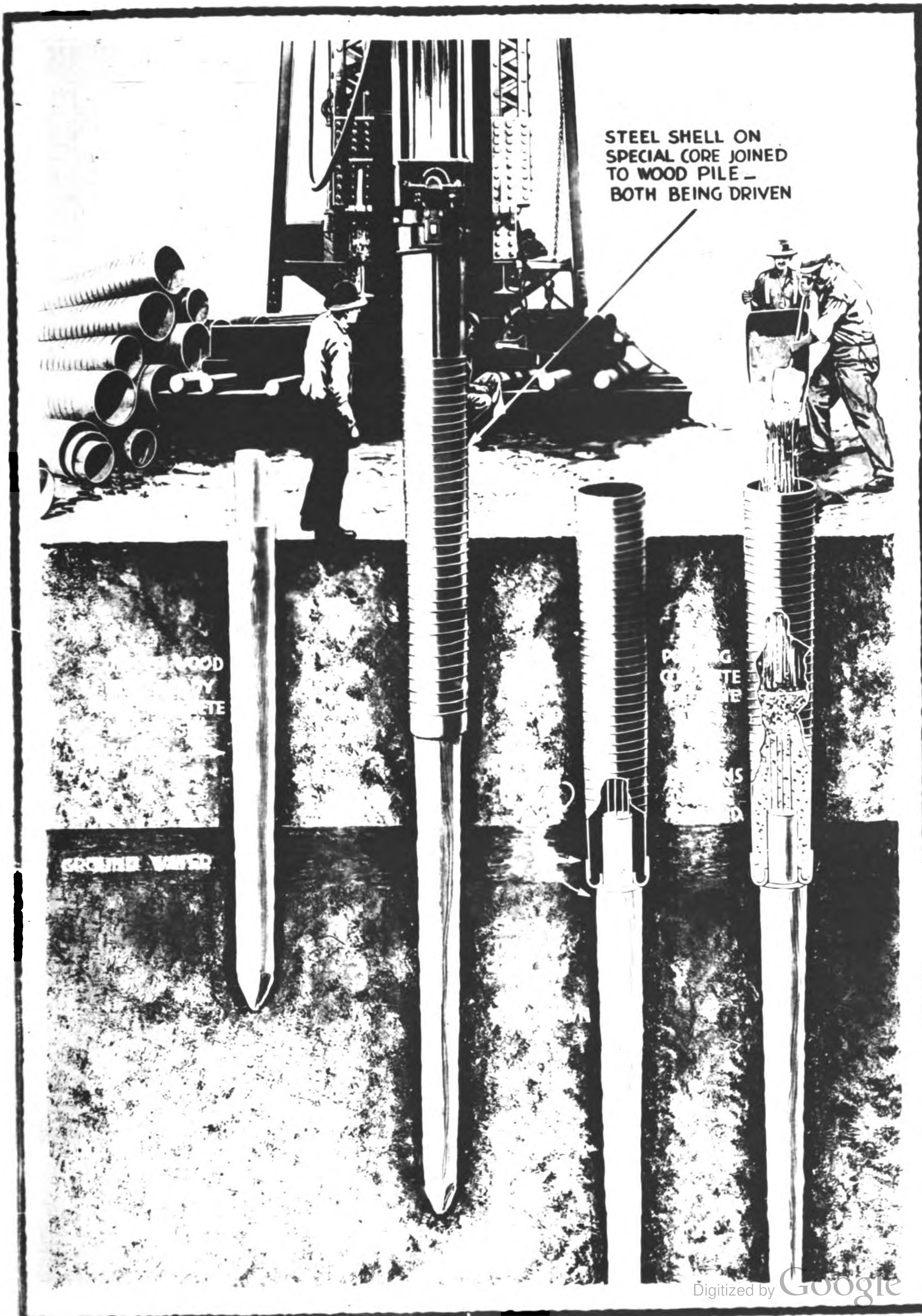
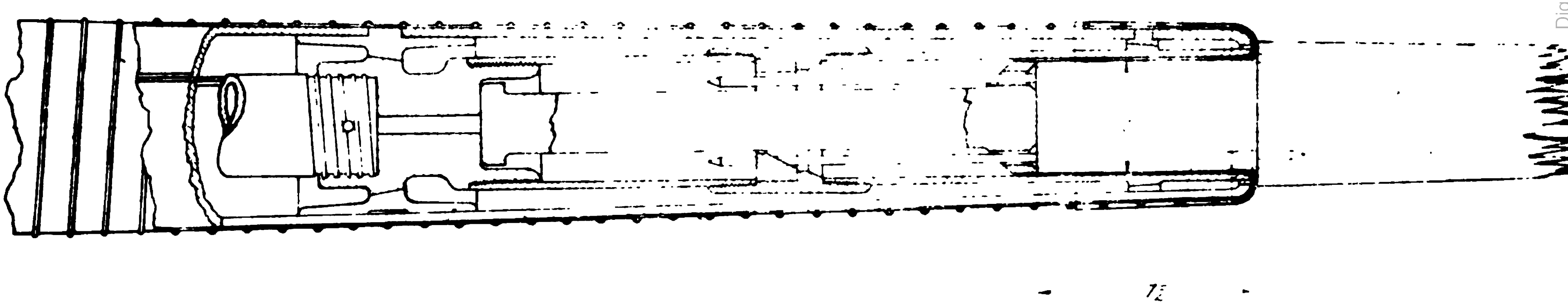

STEEL SHELL ON
SPECIAL CORE JOINED
TO WOOD PILE –
BOTH BEING DRIVEN

PLATE II.



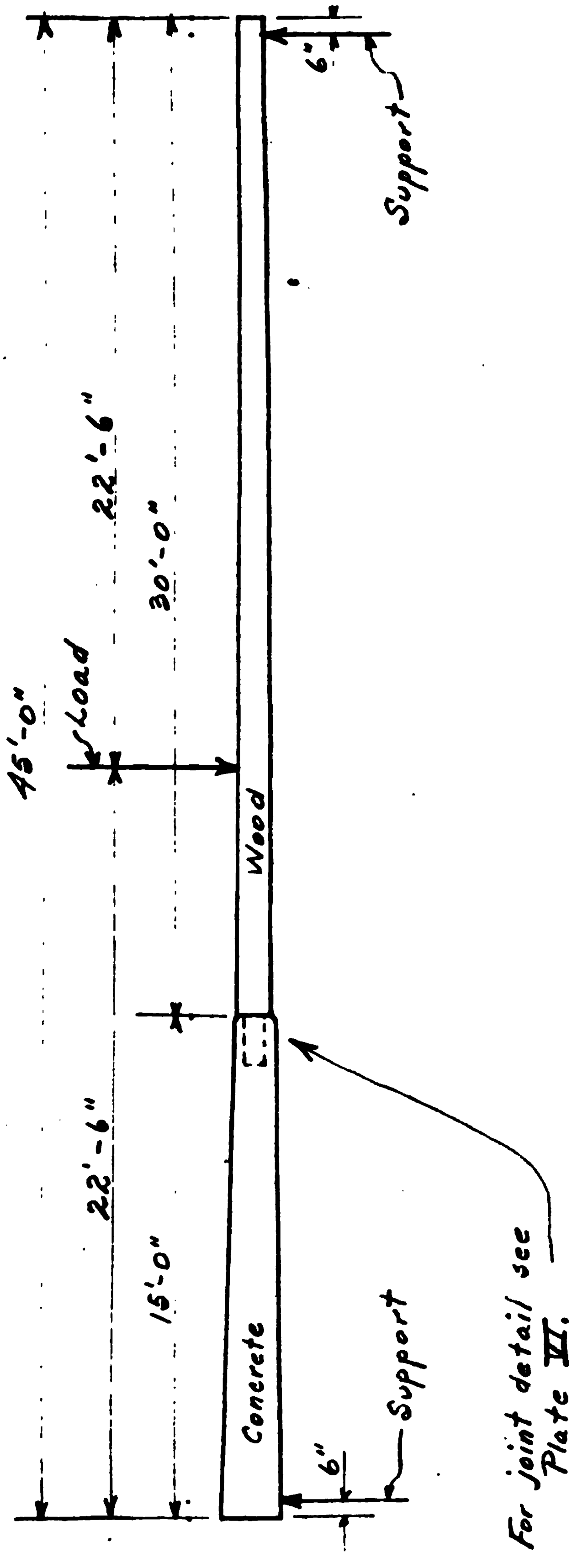
RAYMOND CONCRETE PILE COMP
STREET NEW YORK CIT

METHOD OF DRIVING
COMPOSITE PILES
No. 2

SCALE 1/2" = 12"
DATE 7 29 10
BY L. P.

No. N.Y. 5389

PLATE III.

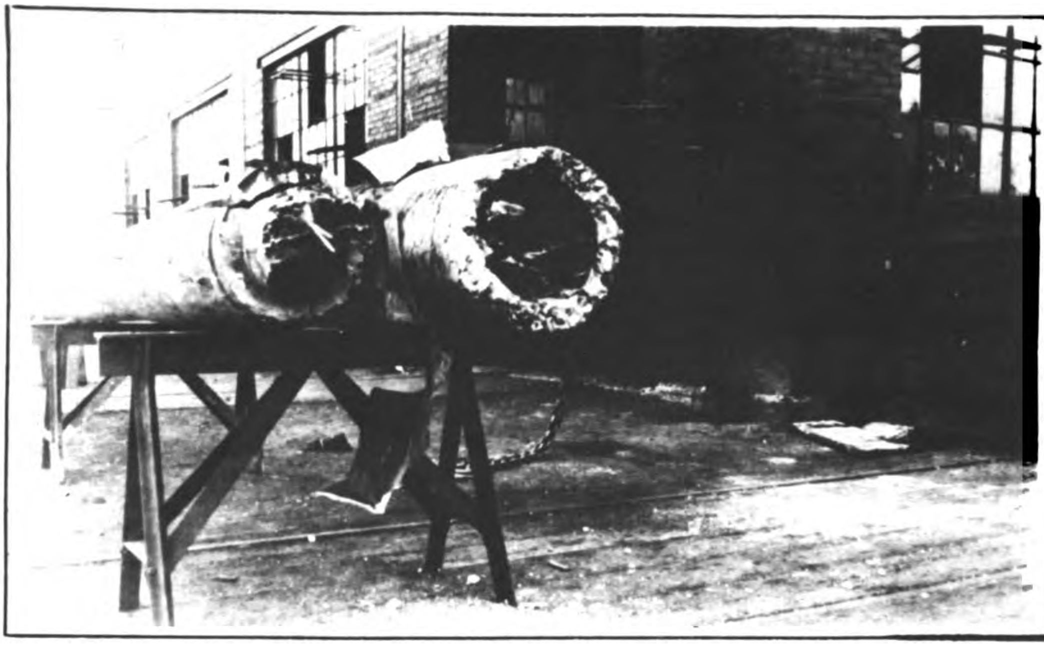


LOADING ARRANGEMENT FOR TESTS A_1, A_2 and A_3

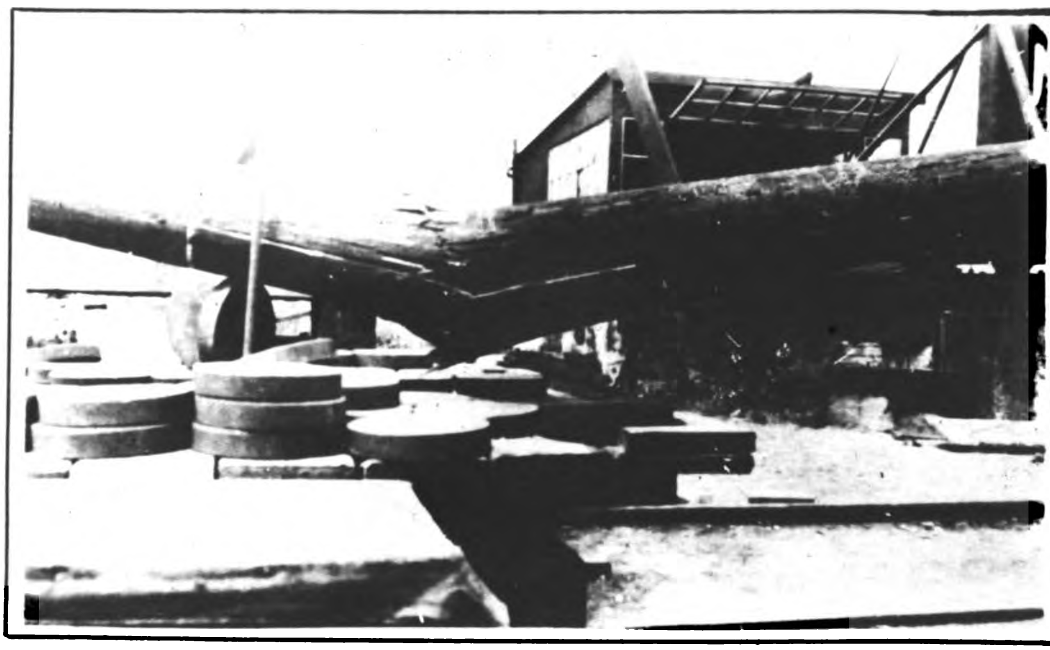
PLATE IV

TEST	LOAD	DEFLECTION	REMARKS
A-1	0 625 1000 1400 1600 1800 2000 2200 2400 2600 2800 3000 3200	2" 3-1/16" 3-9/16" 4-5/16" 4-3/4" 5-1/4" 5-13/16" 6-5/16" 6-15/16" 7-13/16" 8-13/16" 9-11/16"	At 1000 lbs. there was an audible cracking of both joint and wood pile. The load gradually settled from this point, the joint giving and the pile splitting lengthwise from the joint. On the load reaching 3200 lbs. the pile sagged for five minutes when it split lengthwise. It also snapped and broke at the first pin. The pin remained unmoved and unbent. The concrete about the tenon was slightly cracked. Photograph A-1.
A-2	0 625 1000 1200 1400 1600 1800 2000 2200 2400 2600	2-5/16" 3-7/8" 5-3/16" 5-11/16" 6-1/4" 7" 8-5/8" 10" 10-7/8" 12"	There was a noticable weak spot in this wood pile 18" toward the point of pile. The wood began cracking with the weight of the platform alone, 625 lbs., and sagged gradually up to 2800 lbs. which load it held for six minutes before breaking down. At 2400 lbs. there was a slight cracking sound from the joint. However, upon removing the boot and shell, the concrete showed no cracks nor the joint any signs of straining. The pins were intact. The break was a zig zag one, the top of the pile buckling while the lower section pulled apart. Photograph A-2.
A-3.	0 625 1000 1200 1400 1600 1800 2000 2200 2400 2600 2800 3000 3200	2-1/4" 3-7/16" 4-1/16" 4-15/16" 5-9/16" 6-3/4" 7-5/8" 10-1/8" 12-1/8" 13-5/8" 15-1/2" 16-15/16" 18-15/16"	The wood pile here began splitting with the first load, continuing to do so in several places. At 2400 lbs. the joint showed signs of the strain and began to open up on the lower side. At 3200 lbs. the wood pile cracked but held the load for nine minutes before breaking down between point of pile and load, 3' from the latter. Photograph A-3.

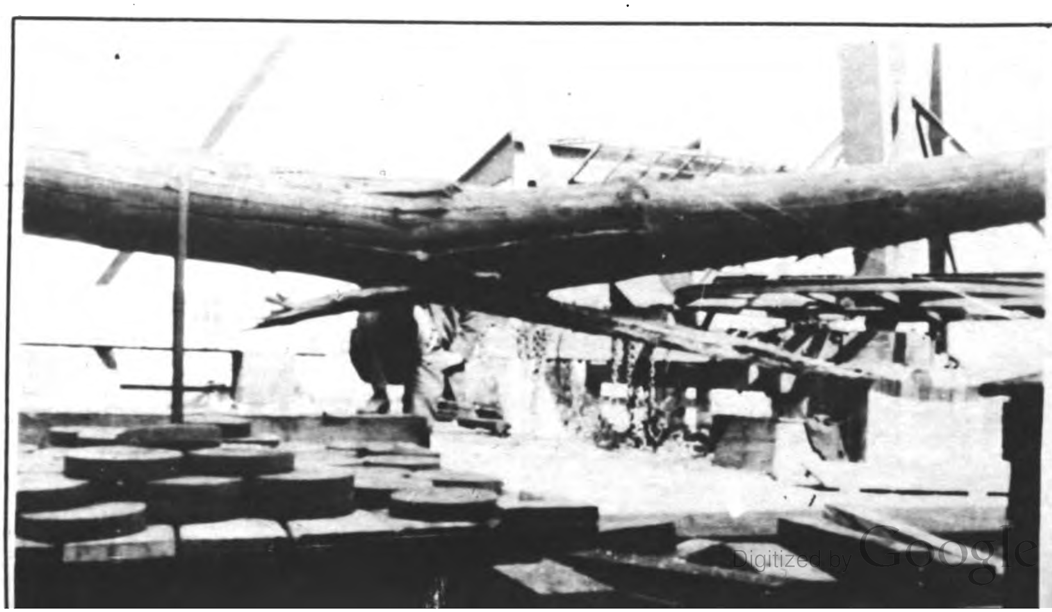
PLATE V.



A1

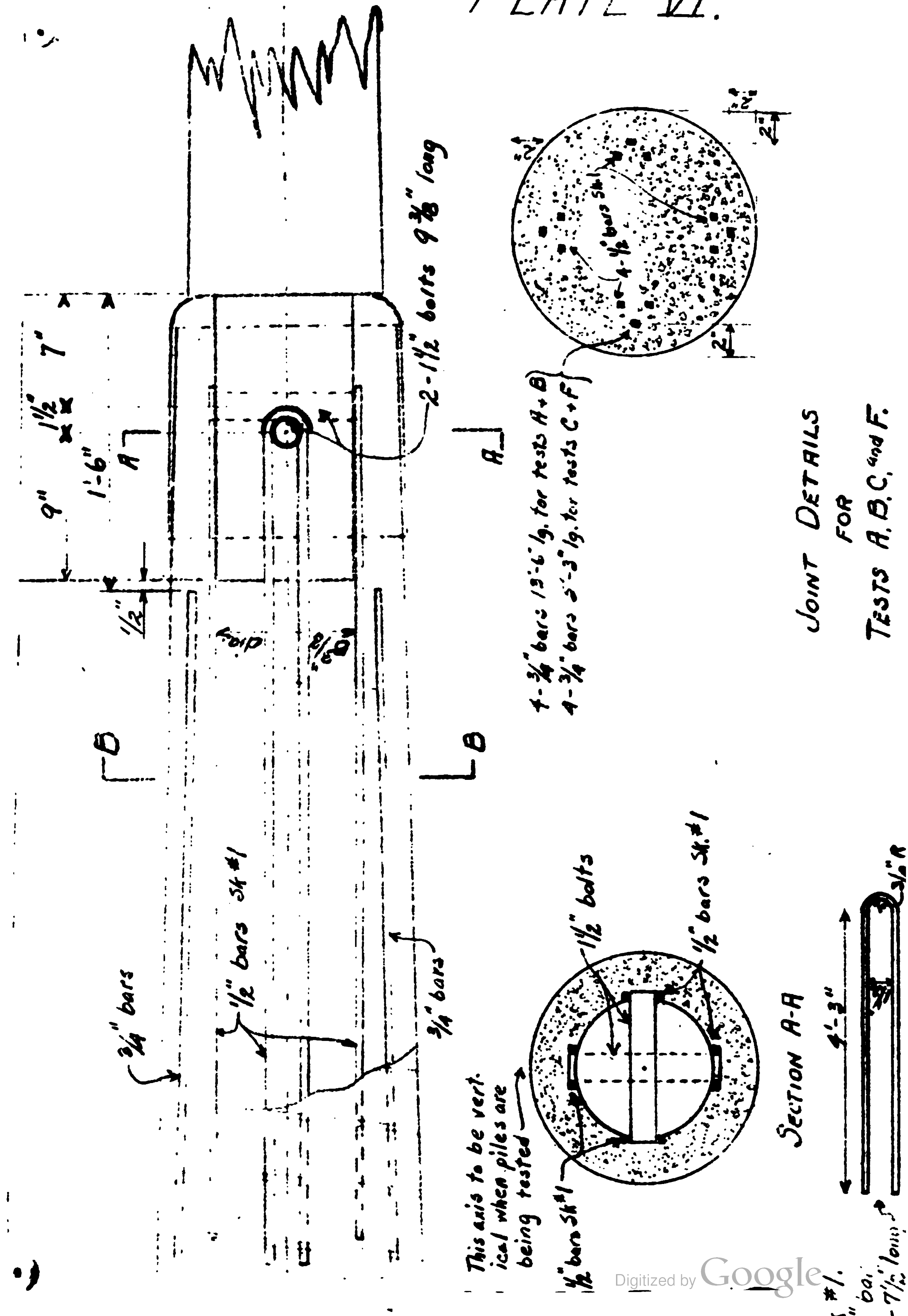


A2



A3

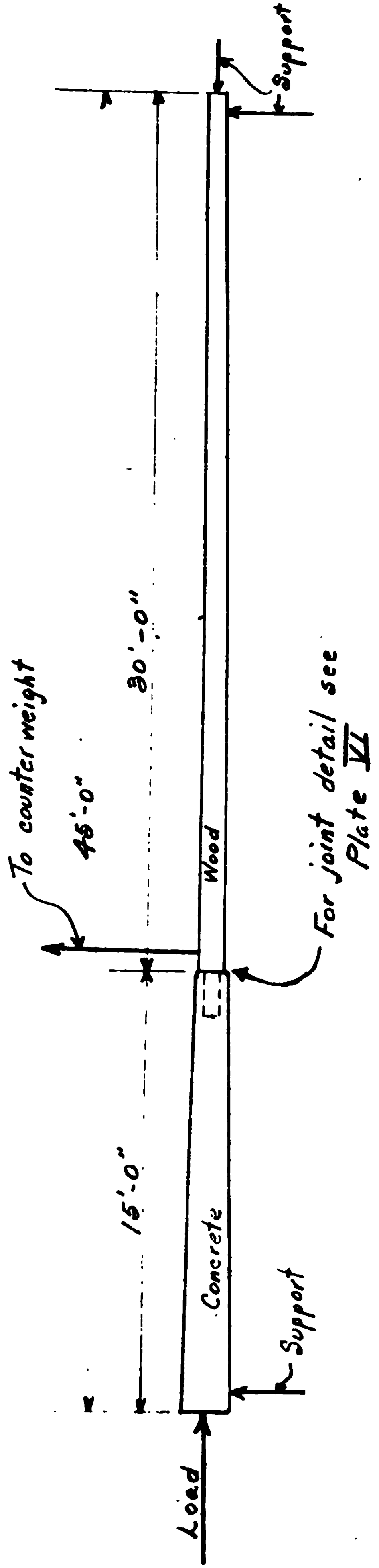
PLATE VI.

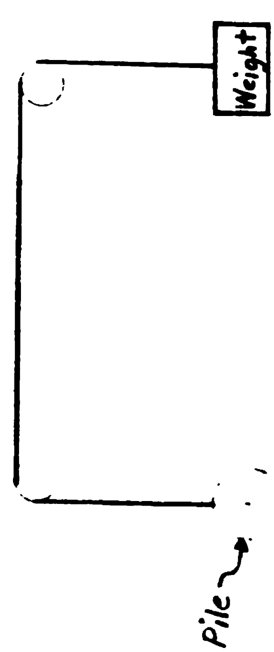


JOINT DETAILS FOR TESTS A, B, C, and F.

PLATE VII

LOADING ARRANGEMENT FOR TEST B.



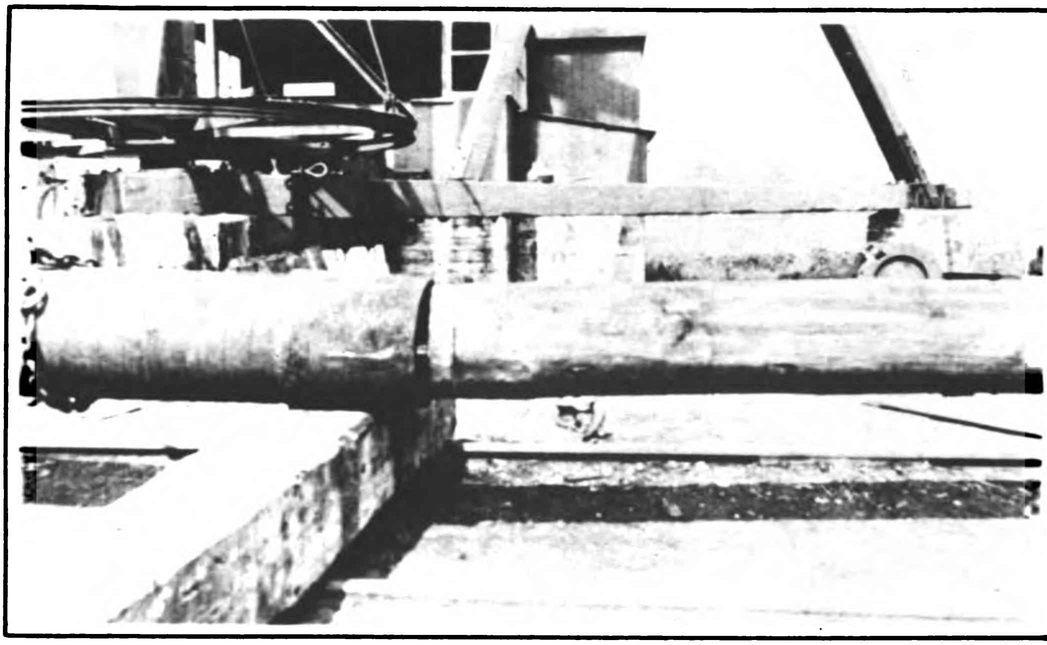


Arrangement of counter weight.

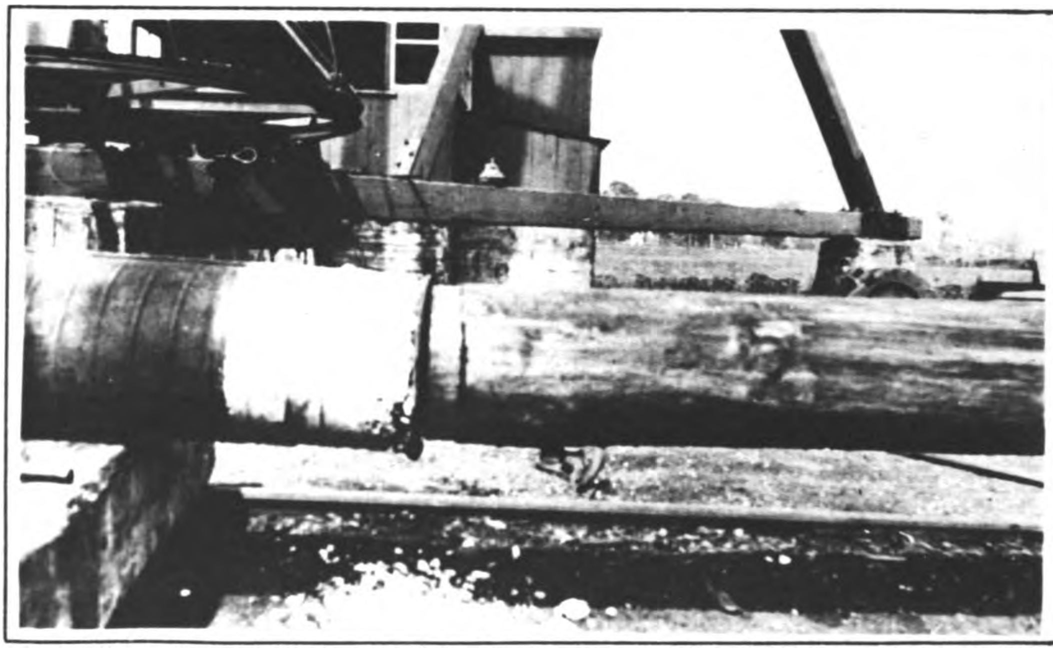
PLATE VIII

TEST	LOAD	DEFLECTION	REMARKS
	0 Tons	2-1/4"	The sag of the pile due to its own weight was 2¼". On being straightened out and 10 tons pressure applied, the pile, unsupported from below, deflected to the right (looking toward the point of the pile) in a horizontal plane. The pile gave gradually and steadily at 29½ tons, the joint opening up and at 36 tons it had opened up 2". The tenon split at the edge of the joint. On removal of the load, the joint sprang back losing 3" of the total deflection, which was 9-9/16". Neither the concrete nor the pins showed signs of the strain. Photos B-1 and B-2 show the joint after it had recovered 3" of the deflection. B-3 shows the pins unmoved and the slight crack which appeared in the tenon. At <u>25 tons</u> there was 1-1/16" deflection and no signs of straining at the joint.
	10 "	3/8"	
	20 "	13/16"	
	25 "	1-1/16"	
B.	28 "	1-7/16"	
	30 "	1-15/16"	
	32 "	2-9/16"	
	34 "	3-9/16"	
	35 "	4-5/16"	
	36 "	9-9/16"	

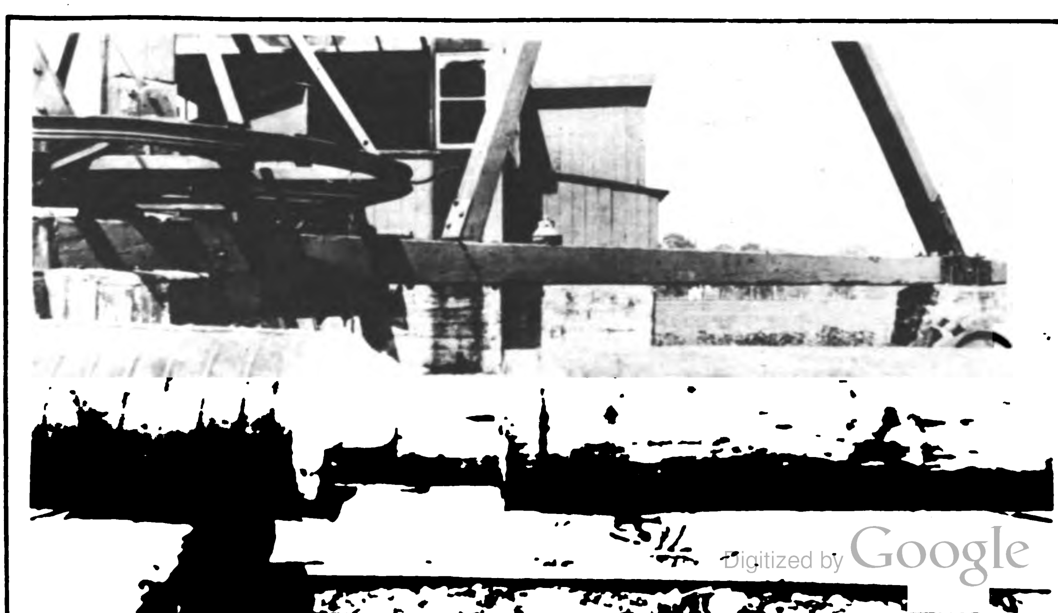
PLATE IX.



B1

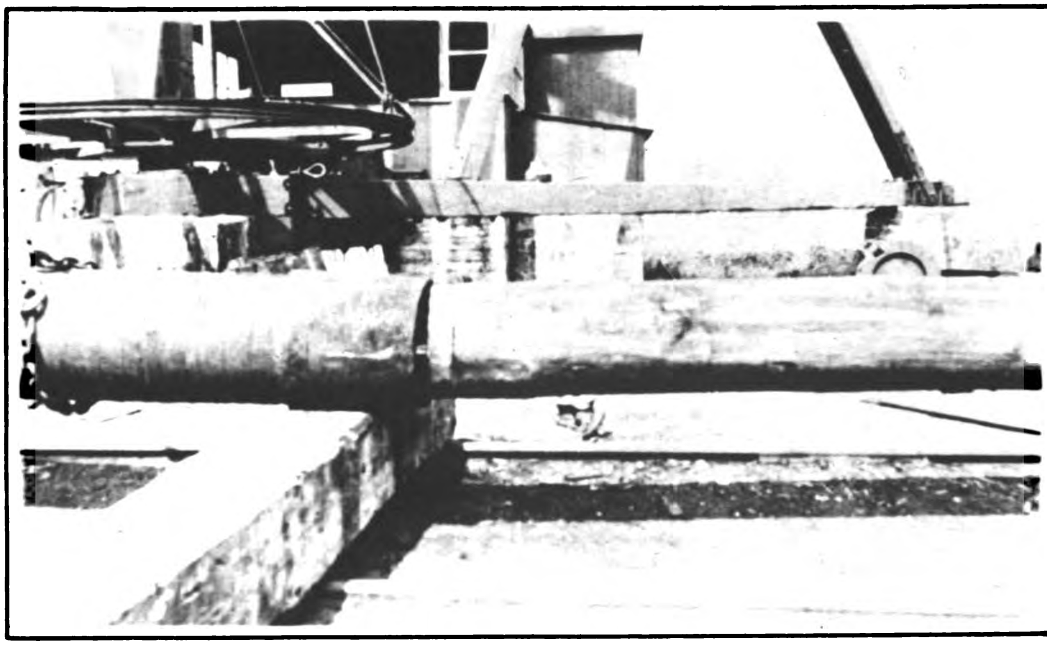


B2

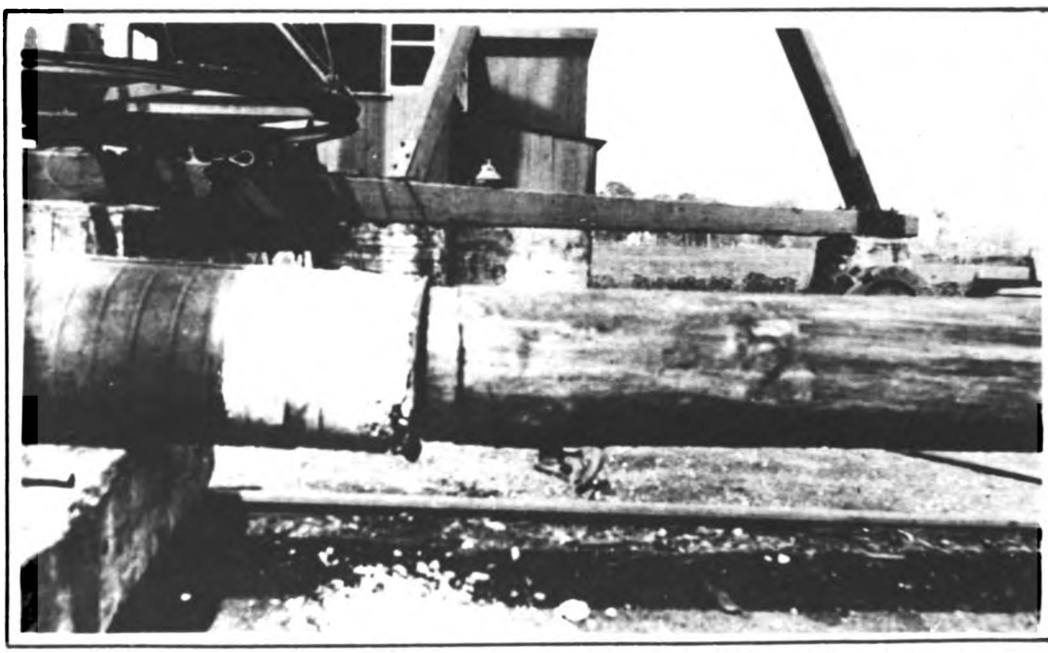


B3

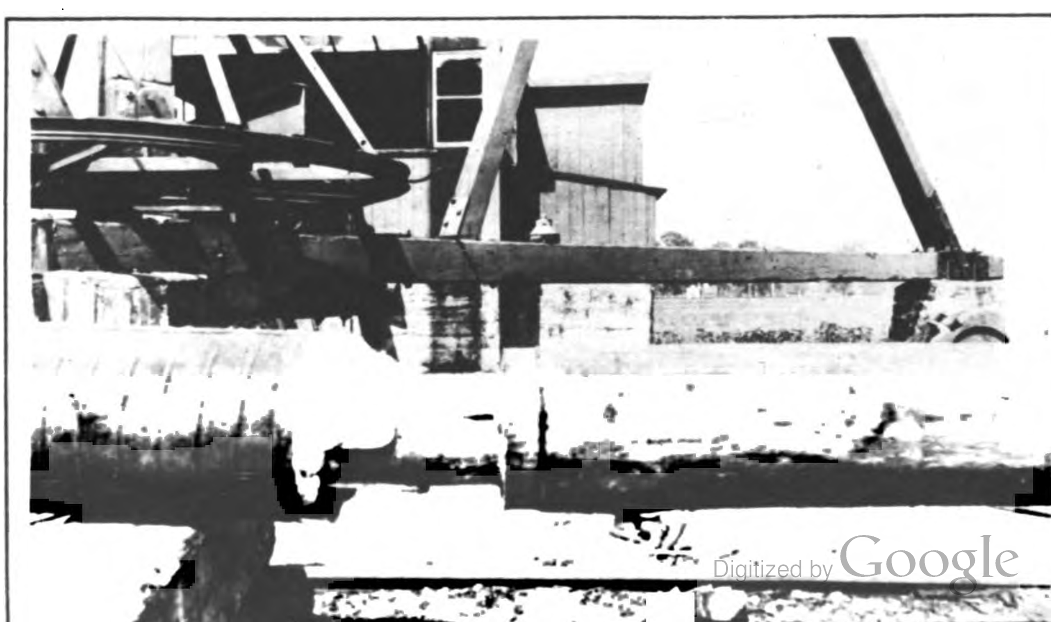
PLATE IX.



B_1

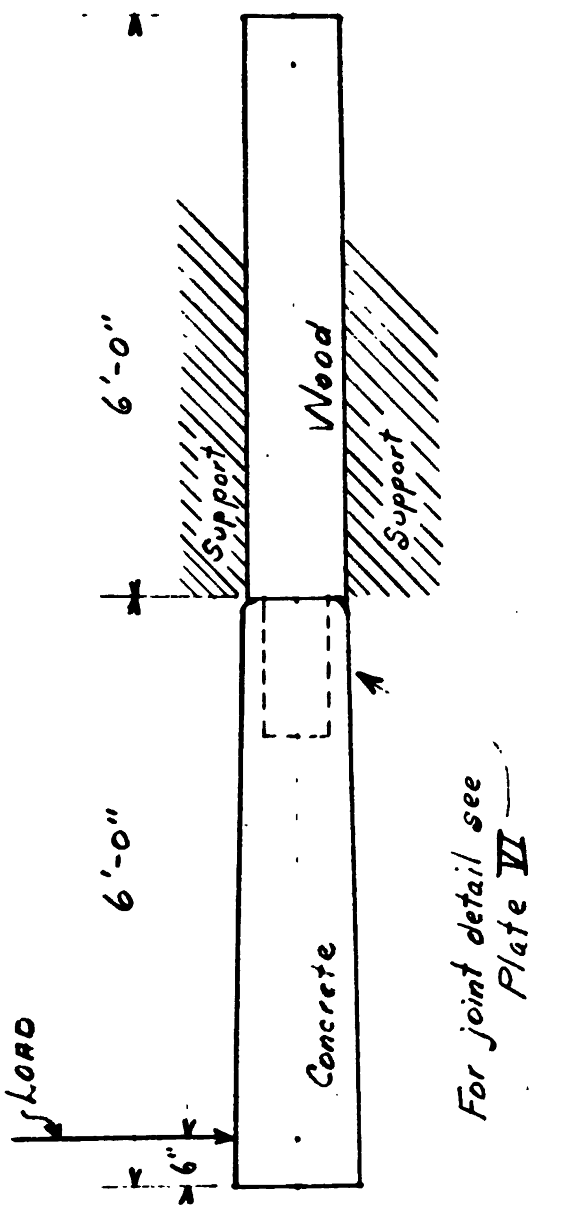


B_2



B_3

PLATE X.

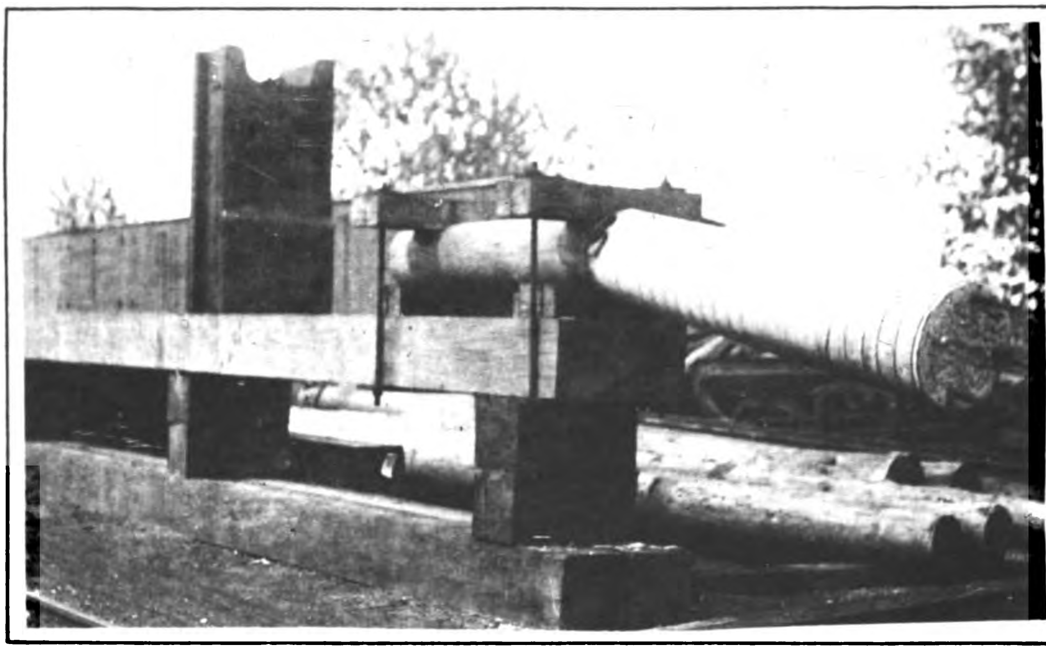


LOADING ARRANGEMENT
FOR
TESTS C_1, C_2, and C_3

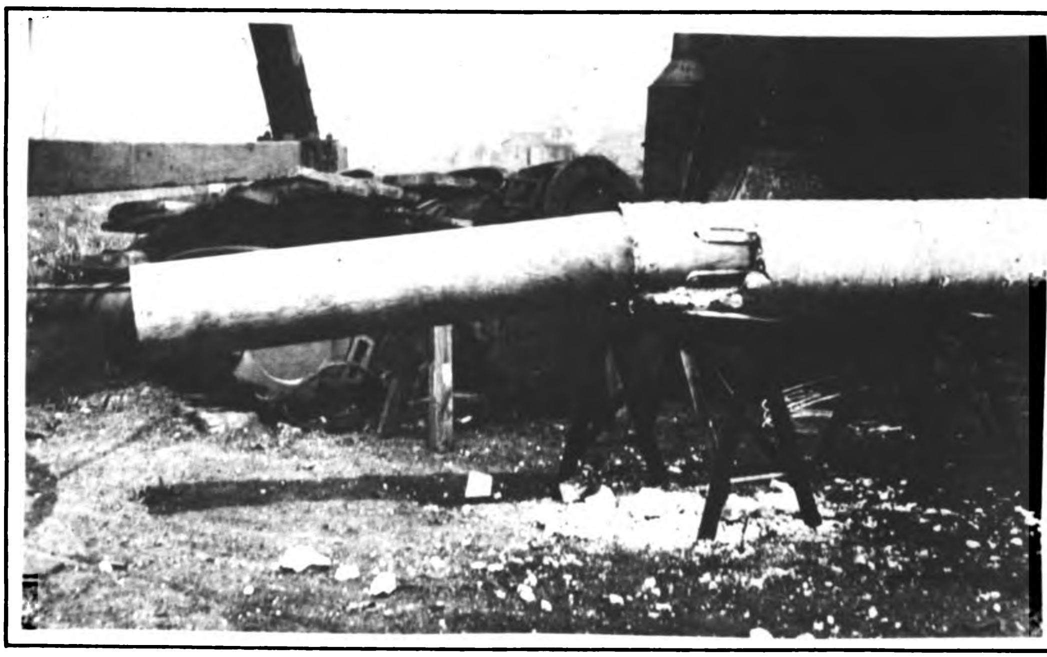
PLATE XI

TEST	LOAD	DEFLECTION	REMARKS
C-1	700	1/8"	The concrete about the joint was slightly cracked. The tenon split, dropping the load, while the pins sheared sections of the tenon but were quite unbent themselves. Photo C-1.
	1150	1/4"	
	1600	3/8"	
	2050	1/2"	
	2500	5/8"	
	2900	3/4"	
	3400	15/16"	
	3850	1-1/8"	
	3950	1-1/4"	
	4650	1-5/8"	
	5450	2-1/8"	
	6050	3-3/16"	
	6400	4-1/16"	
C-2	0	1/8"	The tenon broke here below the pins. There was a very slight deflection in this test. For ten minutes am increased settlement of 5/16" with a load of 4700 lbs. was recorded. 5600 lbs. settled 1/4", then cracked and settled slowly for three minutes. The break was due to the transverse parting of the fibres of the tenon. The pins showed no signs of strain nor did the concrete surrounding the tenon.
	625	1/4"	
	925	3/8"	
	1225	1/2"	
	1600	5/8"	
	2025	3/4"	
	2300	7/8"	
	2625	1-1/16"	
	3000	1-3/16"	
	3200	1-1/4"	
	3425	1-5/16"	
	3650	1-13/16"	
	3900	2-1/8"	
	4300	2-11/32"	
	4700		
	5200		
	5600		
C-3	0	3/32"	There was more promounced deflection of this pile. At 4025 lbs. the concrete cracked audibly. At 6255 lbs. the load gave gradually, breaking down in three minutes. The tenon broke off below the pins. The concrete surrounding the tenon cracked slightly and the pins had started to shear sections out of the tenon but were not bent. Photo C-3 shows the work of the pins which had been driven out for inspection purposes when the photo was taken.
	625	9/32"	
	850	11/32"	
	1300	17/32"	
	1725	23/32"	
	1950	27/32"	
	2400	1-3/32"	
	2600	1-7/32"	
	3000	1-15/32"	
	3200	1-19/32"	
	3400	1-23/32"	
	3600	1-27/32"	
	4025	2-3/32"	
	4400	2-13/32"	
	4800	2-3/4"	
	5200	3-12/32"	
	5600	3-3/4"	
	6025	4-9/32"	
	6250		

PLATE XII.



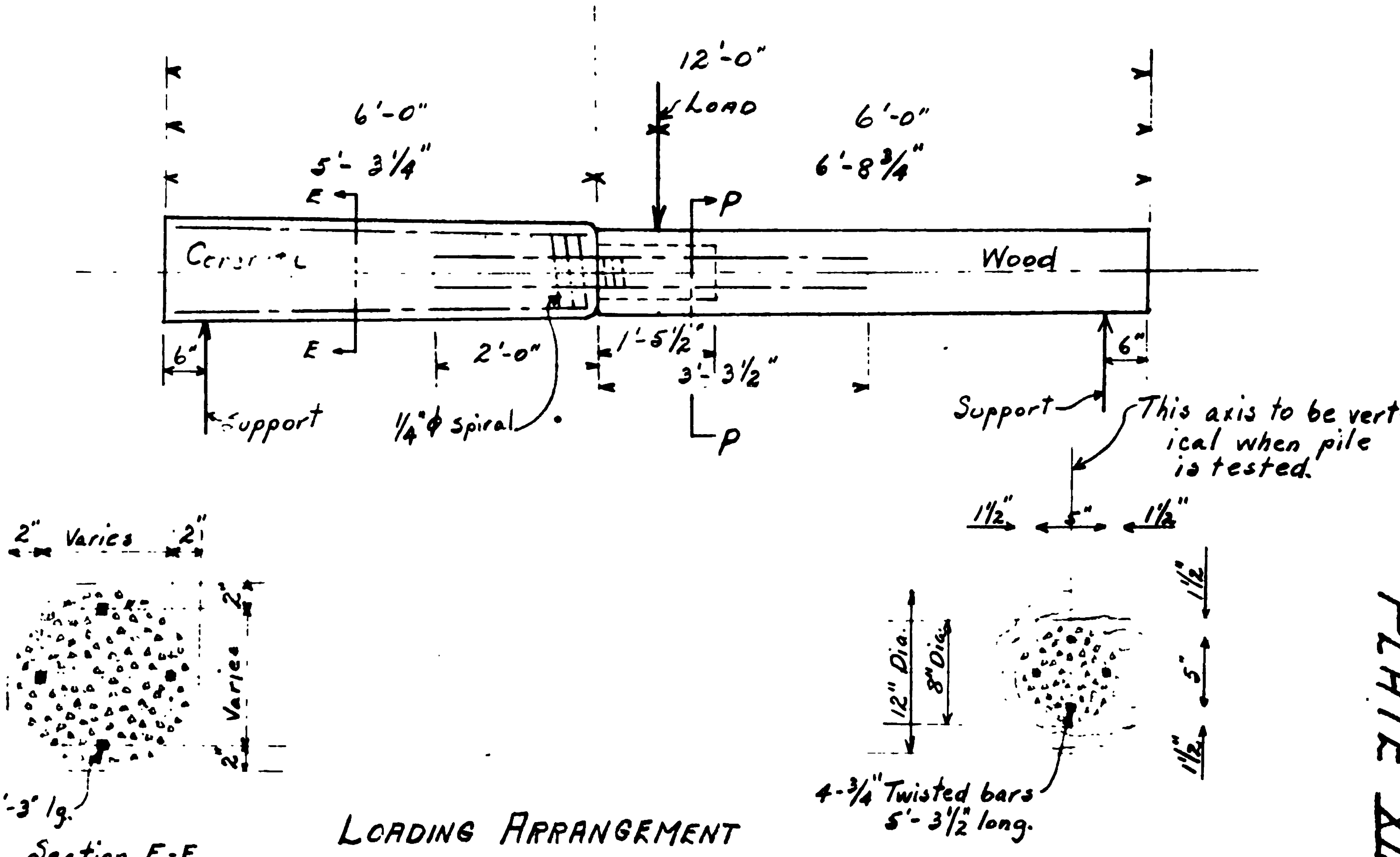
C_1



C_1

C_3

PLATE XIII.

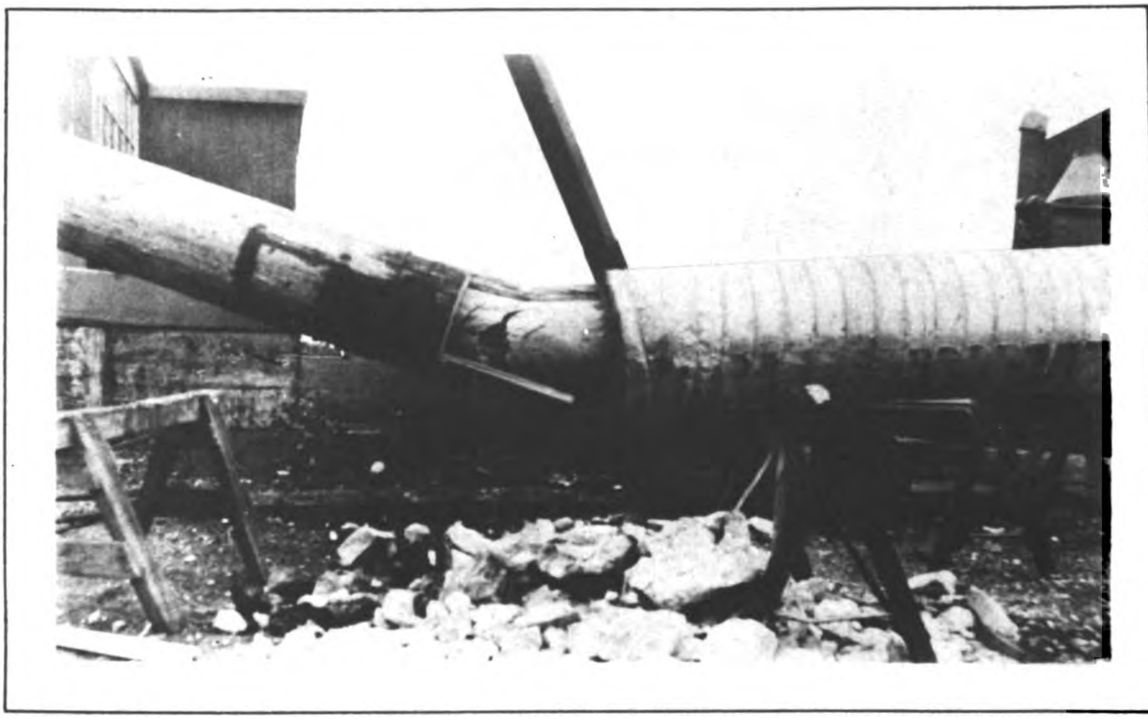


LOADING ARRANGEMENT
FOR
TESTS D_1, D_2, and D_3.

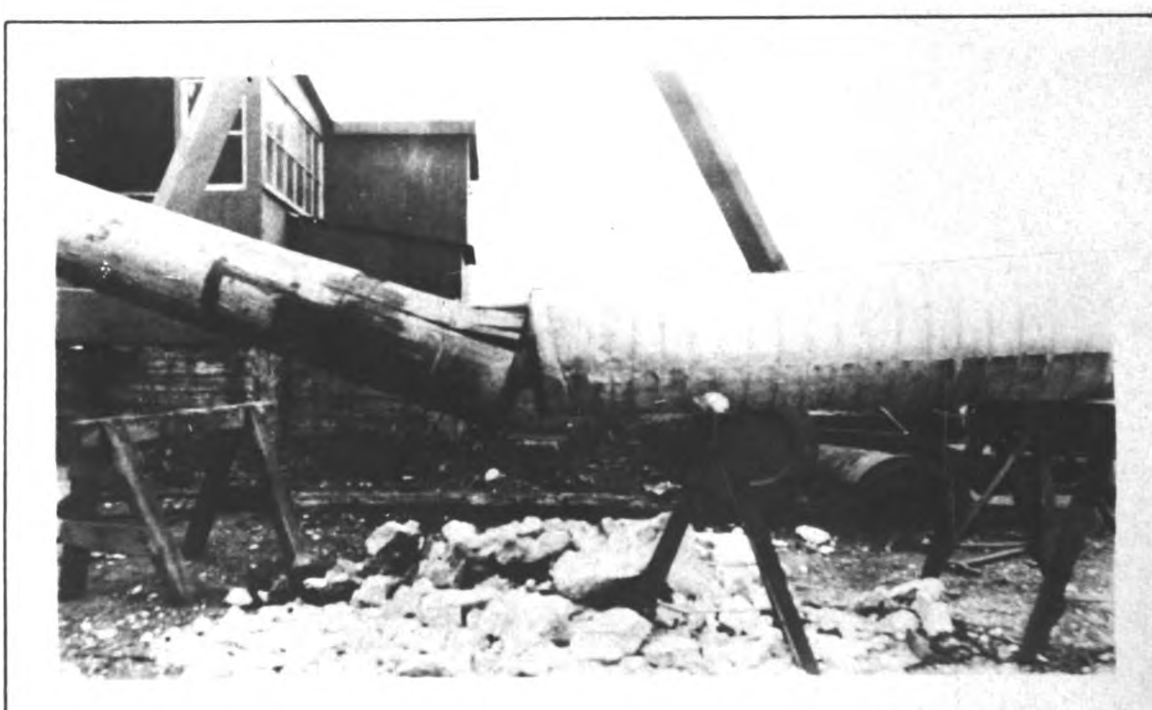
PLATE XIV.

TEST	LOAD	DEFLECTION	REMARKS
D-1	0 625 1400 1900 2500 3100 3800 4300 4800 5300 5800	1/32" 3/32" 1/8" 5/32" 3/16" 7/32" 1/4" 5/16" 1/2" 3/4"	This type of joint was very stiff. It held up almost to the breaking point with but a slight deflection. The concrete tenon cracked audibly, the wood surrounding the tenon giving way immediately. The concrete tenon cracked slightly throughout. The bars were bent and had pulled out of the wood 3/4" but had not been separated from the concrete. The load sustained was 5300 lbs., the total deflection being 3/4". At 5800 lbs. the pile broke. Photo D-1.
D-2	0 625 1400 2000 2700 3200 3700 4200 4700 5200 5700 6200 6700 6900	1/32" 3/32" 5/32" 7/32" 11/32" 7/16" 19/32" 13/16" 1-1/16" 1-1/2" 1-15/16" 2" 4"	The sagging began earlier in this test and was quite gradual but the break came at the same point on the tenon as in D-1, with the difference that the concrete tenon here had crumbled throughout. The load sustained was 6700 lbs. at a total deflection of 4", the breaking point being 6900 lbs. The bars were bent and had been drawn from the wood but their bond with the concrete remained unbroken.
D-3	0 625 1400 2000 2700 3200 3700 4200 4700 5200 5700	1/32" 1/32" 3/16" 7/32" 1/4" 9/32" 5/16" 3/8" 1-1/8" 1-3/8"	At 2000 lbs. the section of the wood pile encasing the concrete tenon began to split and at 5700 lbs. the tenon snapped in a clean break, dropping the load. This was a stiff pile. The load sustained was 5200 lbs., the total deflection being 1-3/8". The bars were very slightly bent and had been pulled out of the wood 1/2" but remained fast in the concrete. Photo D-3.

PLATE XV.

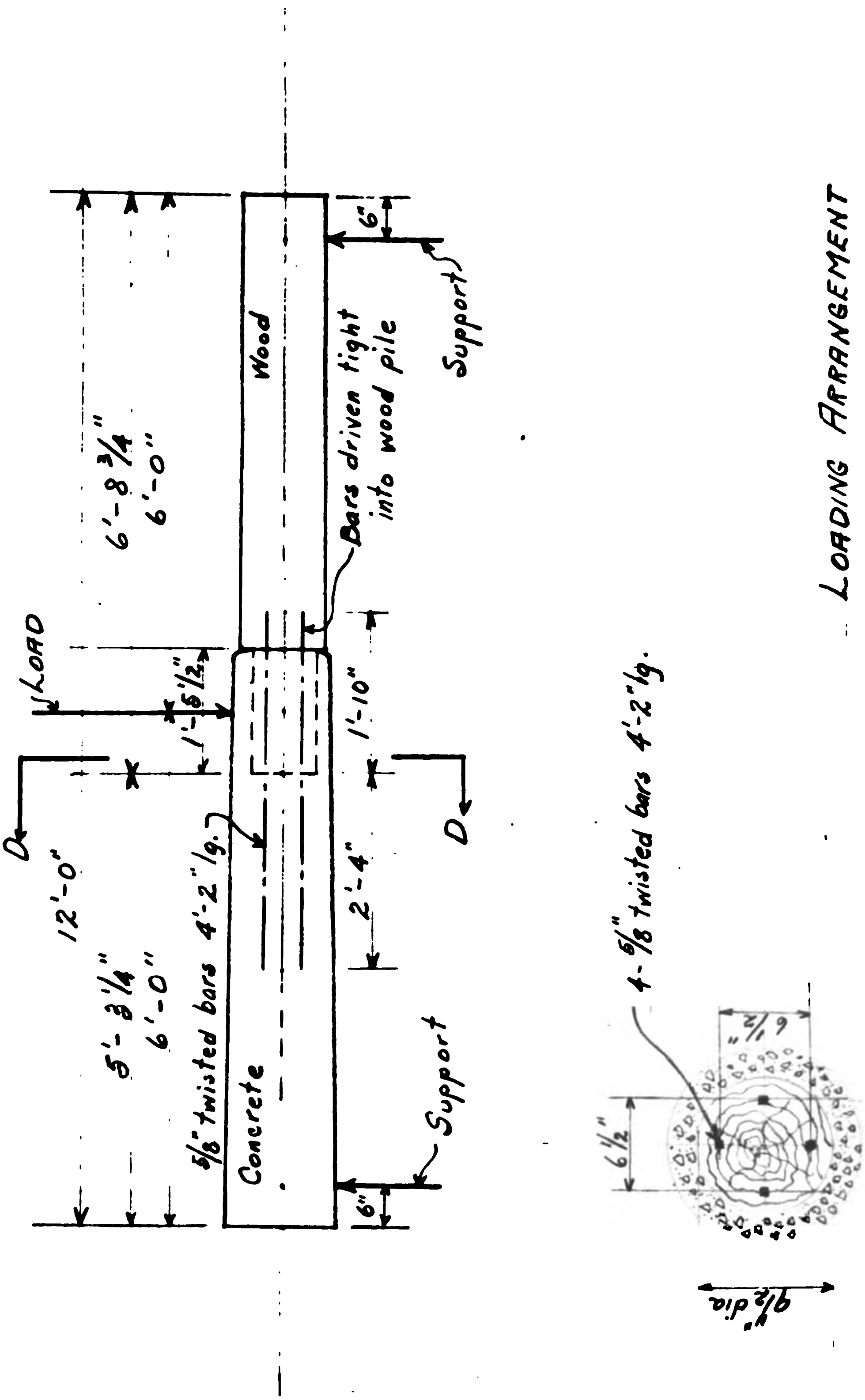


D



D

PLATE XVI.



LOADING ARRANGEMENT FOR TESTS E_1, E_2, and E_3

PLATE XVII

TEST	LOADS	DEFLECTION	REMARKS
E-1	0 625 1400 2000 2600 3200 4400 5300 6300 6800 7800 8800 9800 10800 11800 12800 13700 14000	1/32" 3/32" 3/32" 5/32" 3/16" 3/8" 1/2" 19/32" 29/32" 1-1/32" 1-1/4" 1-15/32" 1-23/32" 2-5/32" 2-13/32" 2-23/32" 3-1/32" 3-3/16"	This test produced the individual record load of 14000 lbs. supported with a total deflection of 3-3/16". At 14,500 lbs. the concrete broke at the end of the wood tenon. The concrete parted cleanly with but a few slight cracks at the edge of the break. Nowhere else was there the slightest sign of a strain. The rods pulled from the wood and bent. Photo E-1.
E-2	0 625 1400 2000 2500 3000 3600 4200 5600 6500 7500 8500 9500 10500 11500 12000	1/32" 1/16" 3/16" 1/4" 5/16" 11/32" 13/32" 15/32" 3/4" 7/8" 1-3/32" 1-13/32" 1-25/32" 2-1/16" 2-15/16"	The break came here at the same point on pile as in the preceding test. The rods pulled out of the wood and bent. The concrete was cracked slightly about the break and the shell ripped open in line of the break at the edge of the boot. The load supported was 11500 lbs. deflection 2-15/16". 12000 lbs. sagged for 10 minutes before joint finally broke down. Photo E-2.
E-3	0 625 1400 2000 2600 3200 4400 5500 6400 7400 8400 9400 10400 10900 11000	1/32" 1/16" 3/32" 1/8" 3/16" 7/32" 5/16" 5/8" 13/16" 1-1/16" 1-15/32" 1-31/32" 2-27/32" 3-19/32"	At 5500 lbs. the joint was heard to crack but nothing could be seen. 8400 lbs. brought 1-15/32" deflection but a load of 10,900 lbs. held steadily at 3-19/32" settlement for five minutes. 11,000 lbs. however, snapped the pile suddenly at the end of the wood tenon, the shell ripped open along the seam from the joint, the concrete at the break crumbled and the rods pulled out of the wood and bent. Photo E-3.

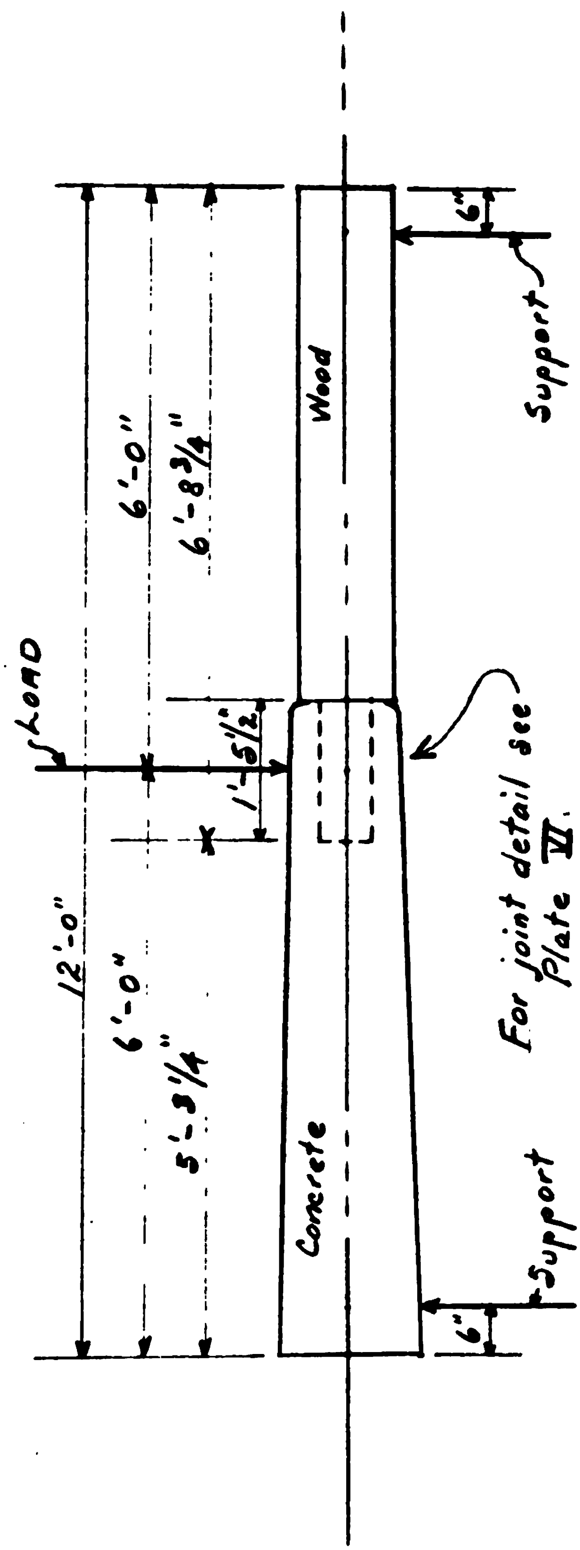
PLATE XVIII.

E_1

E_2

E_3

PLATE XIX.

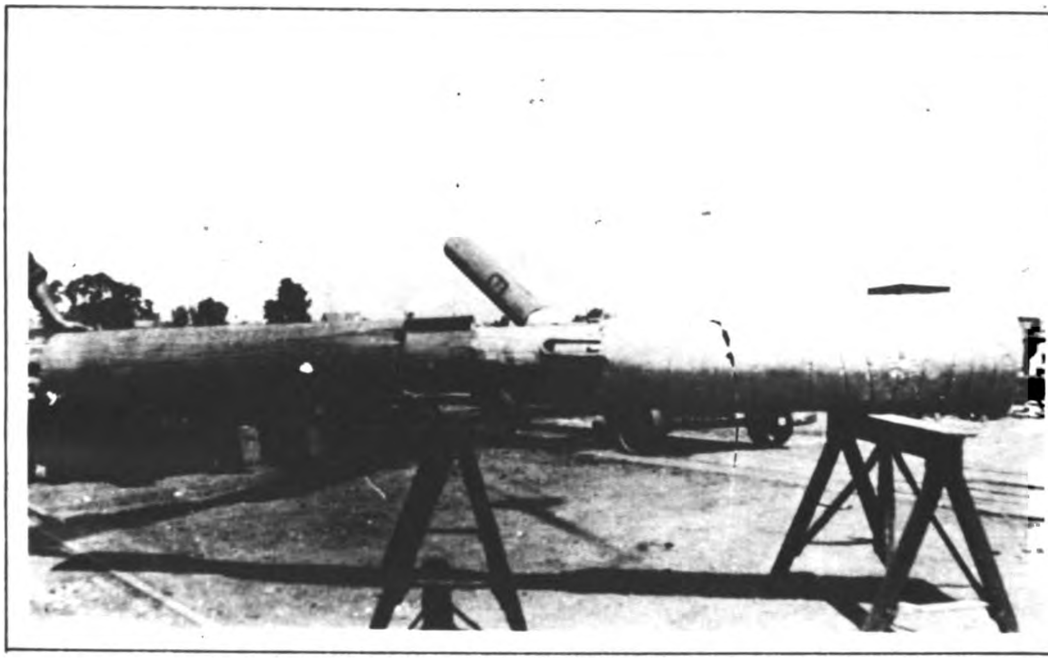


LOADING ARRANGEMENT FOR TESTS F_1, F_2, and F_3.

PLATE XX

TEST	LOAD	DEFLECTION	REMARKS
F-1	0	1/16"	At 8500 lbs. the joint showed signs of the strain and began to open up underneath. 10700 lbs. sagged 4-1/16" for 10 minutes, then stopped. 11725 lbs. sagged gradually for 10 minutes when the tenon cracked, breaking off short below the boot. The concrete, bars and pins were quite intact. Photo F-1.
	625	3/16"	
	1000	1/4"	
	1400	5/16"	
	2200	7/16"	
	3000	5/8"	
	3800	7/8"	
	4650	1-1/8"	
	5500	1-3/8"	
	6400	1-5/8"	
	6850	1-3/4"	
	7700	2-1/32"	
	8500	2-3/8"	
	9600	2-15/16"	
	10700	4-1/16"	
	11050	5-1/16"	
	11525	7-1/8"	
	11725		
F-2	0	1/16"	This was a very stiff joint. 14360 lbs. sagged 1/8" before tenon cracked at the end of the boot, where it hung for 10 minutes before breaking down. The load sustained was 13,400 lbs.; total deflection 1-29/32". On stripping the joint, the tenon was foundto have pulled apart $1\frac{1}{2}$". The pins sheared off sections of the tenon but were quite unbent, the concrete about the tenon showing only very slight cracks. This break is illustrated, only in a little more pronounced degree, in the Photos F-3.
	625	3/16"	
	1000	1/4"	
	1700	11/32"	
	2300	15/32"	
	3300	21/32"	
	4100	3/4"	
	5300	29/32"	
	6100	1-1/16"	
	6900	1-3/16"	
	8100	1-9/32"	
	9700	1-1/2"	
	10500	1-9/16"	
	11500	1-21/32"	
	12900	1-25/32"	
	13400	1-29/32"	
	14360		
F-3	0	1/16"	The load sustained was 13000 lbs. total deflection 3-5/32". When loaded to 13500 lbs. the tenon cracked several times and broke below the pins 4" from the point of the concrete. The pins sheared off sections of the tenon for $3\frac{1}{2}$" but remained unbent. The concrete showed no signs of strain in spite of the fact that the tenon was practically pulled apart. Photo F-3.
	625	3/16"	
	1400	7/16"	
	2600	11/16"	
	3900	13/16"	
	5500	1"	
	6500	1-1/16"	
	7500	1-5/16"	
	8500	1-13/32"	
	9500	1-9/16"	
	11500	1-29/32"	
	12000	2-9/32"	
	12500	2-25/32"	
	13000	3-5/32"	
	13500		

PLATE XXI.



F_1

F_3

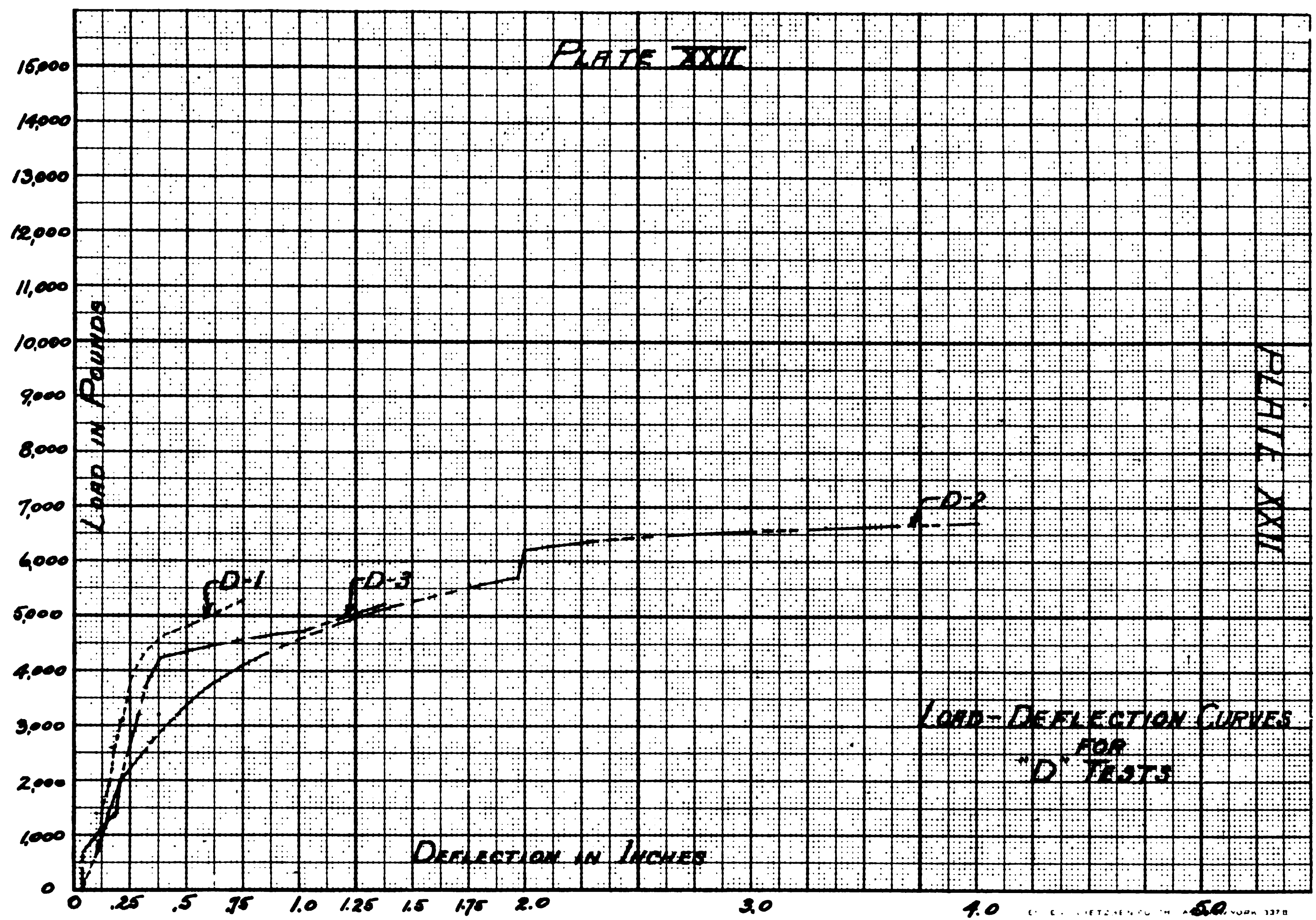


PLATE XXII
15,000
14,000
13,000
12,000
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
LOAD IN POUNDS
D-1
D-3
D-2
0
.25
.5
.75
1.0
1.25
1.5
1.75
2.0
3.0
4.0
5.0
DEFLECTION IN INCHES
LOAD-DEFLECTION CURVES
FOR
"D" TESTS
PLATE XXII

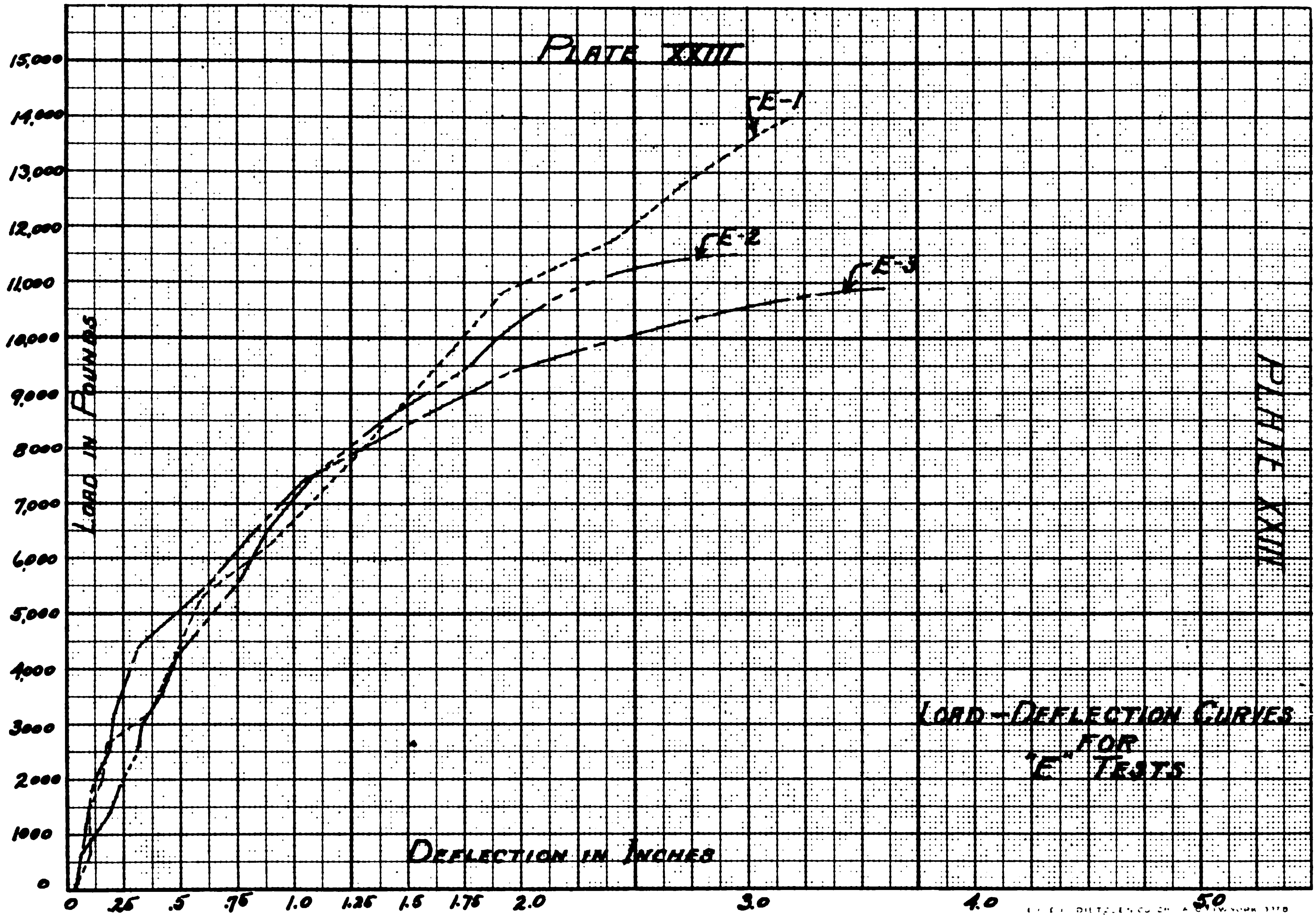


PLATE XXIII
LOAD IN POUNDS
15,000
14,000
13,000
12,000
11,000
10,000
9,000
8000
7,000
6,000
5,000
4000
3000
2000
1000
0
E-1
E-2
E-3
DEFLECTION IN INCHES
0
.25
.5
.75
1.0
1.25
1.5
1.75
2.0
3.0
4.0
5.0
LOAD-DEFLECTION CURVES
FOR
"E" TESTS
PLATE XXIII

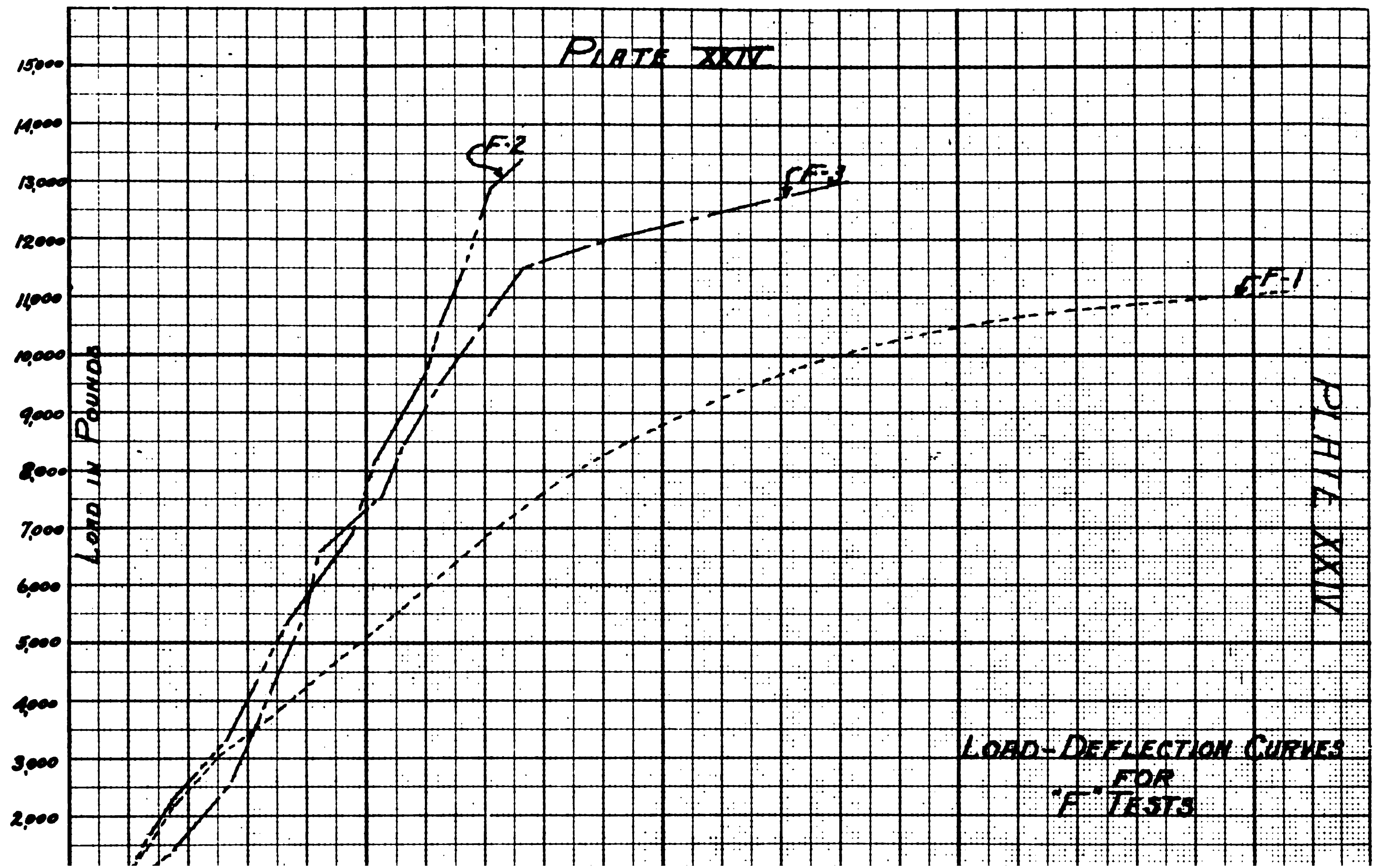


PLATE XXIV
LOAD IN POUNDS
15,000
14,000
13,000
12000
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
F-2
F-3
F-1
PLATE XXIV
LOAD-DEFLECTION CURVES
FOR
"F" TESTS

PLATE XXV.

Column Number				1	2	3	4	5	6	7	8	9	10
Test Letter	Test Number	Type of Test	Manner of Failure	Bend. Mom. on Joint Inch-Lbs.	Fibre stress on wood at base of tenon #/in²	Fibre stress on wood net section thru pin. #/in²	Fibre stress on concrete at end of tenon. #/in²	Average Fibre stress in bars. #/in²	Actual stress in bar. #	Bend. Mom on Pin Inch-Lbs	Fibre stress in Pin #/in²	Shear on wood by pin. #/in²	Bond stress between bar and wood. #/in²
A	1	45' Beam	Wood broke at tenon	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	28400	7,100	21,410	823	
	2	"	[illegible]	[illegible]	[illegible]	[illegible]	317	[illegible]	34550	6,137	[illegible]	711	
	3	"		[illegible]	[illegible]	6250	[illegible]	[illegible]	28400	7,100	21,410	823	
B	1	45' Column	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	787	
C	1	6' Cantilever	[illegible]	[illegible]	7340	10500	[illegible]	95400	47700	11,975	36,100	1352	
	2	"	[illegible]	[illegible]	5450	[illegible]	1235	77400	35700	9,6[illegible]	29,300	1120	
	3	"	"	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]750	44575	[illegible]	[illegible]	1299	
D	1	12' Beam	[illegible]	[illegible]	[illegible]		3480	[illegible]	[illegible]				527
	2	"	[illegible]	[illegible]	[illegible]		4400	[illegible]	44000				667
	3	"	[illegible]	[illegible]	[illegible]		3430	[illegible]	54200				519
E	1	12' Beam	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]				1287
	2	"	"	290,000	[illegible]		1423	[illegible]	59,200				1159
	3	"	"	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]				1005
F	1	12' Beam	[illegible]	[illegible]	[illegible]	[illegible]	1427	[illegible]	[illegible]	[illegible]	[illegible]	1240	
	2	"	[illegible]	[illegible]	[illegible]	11030	1660	[illegible]	[illegible]	12,157	27,700	1415	
	3	"	"	[illegible]	7430	[illegible]	1665	[illegible]	45250	[illegible]	36,450	1399	

Methods of Calculation and Formulas
For
PLATE XXV.

Column 1.

A-Tests $M = \frac{PL}{4} \times \frac{13}{22}$

B-Test $M = \text{Load} \times \text{Deflection}\ (70{,}000^{\#} \times 4\tfrac{5}{16}'')$

C-Tests $M = PL$

D-Tests $M = \frac{PL}{4}$

E-Tests $M = \frac{PL}{4}$

F-Tests $M = \frac{PL}{4}$

Column [illegible].

G[illegible] [illegible] Form[illegible] [illegible] Stress $= \frac{\text{Moment of column 1.}}{\text{Section Modulus.}}$

For A, B, C, and F tests. [illegible] $= .098\,d^3$ [d [illegible] $8\tfrac{3}{8}''$]

For D-Tests [illegible] Mod. $= .098\left(\frac{d^4 - d_1^4}{12}\right)$ [$d = 12''$ and $d_1 = 8''$]

For [illegible] Sec. Mod. $= .098\,d^3$ [$d = 4\tfrac{1}{2}''$]

Column [illegible].

[illegible] Form[illegible] Stress $=$ [illegible] of column 1. [illegible]

Section [illegible] $= .098\,d^3 -$ [illegible] [$d = 8$[illegible] [illegible] $t = 1\tfrac{1}{2}''$]

[illegible] 4.

Stress $= \frac{\text{[illegible]}}{\text{Section [illegible]}}$

Section Mod[illegible] $=$ [illegible] [$d = 13.1''$ [illegible]]

[$d =$ [illegible] $8''$ [illegible]]

Column 5.

$$\text{Stress} = \frac{\text{Moment of column 1.}}{\text{Section Modulus}}$$

Section Modulus $= \frac{I}{c}$ [where c = distance to center of bar from neutral axis of pile]

$I = I_1 + Ar^2$ [where A = area of section and $r = c$ as above]

Column 6.

Stress = Column 5 X Area of bar.

Column 7.

Moment $= \frac{WL}{2}$ [where W = column 6 and $L = 1/2"$]

Column 8.

$$\text{Stress} = \frac{\text{Moment of column 7}}{\text{Section Modulus}}$$

Section Modulus = .098 d^3 [$d = 1 1/2"$]

Column 9.

Load on Pin = 2 x Column 6.

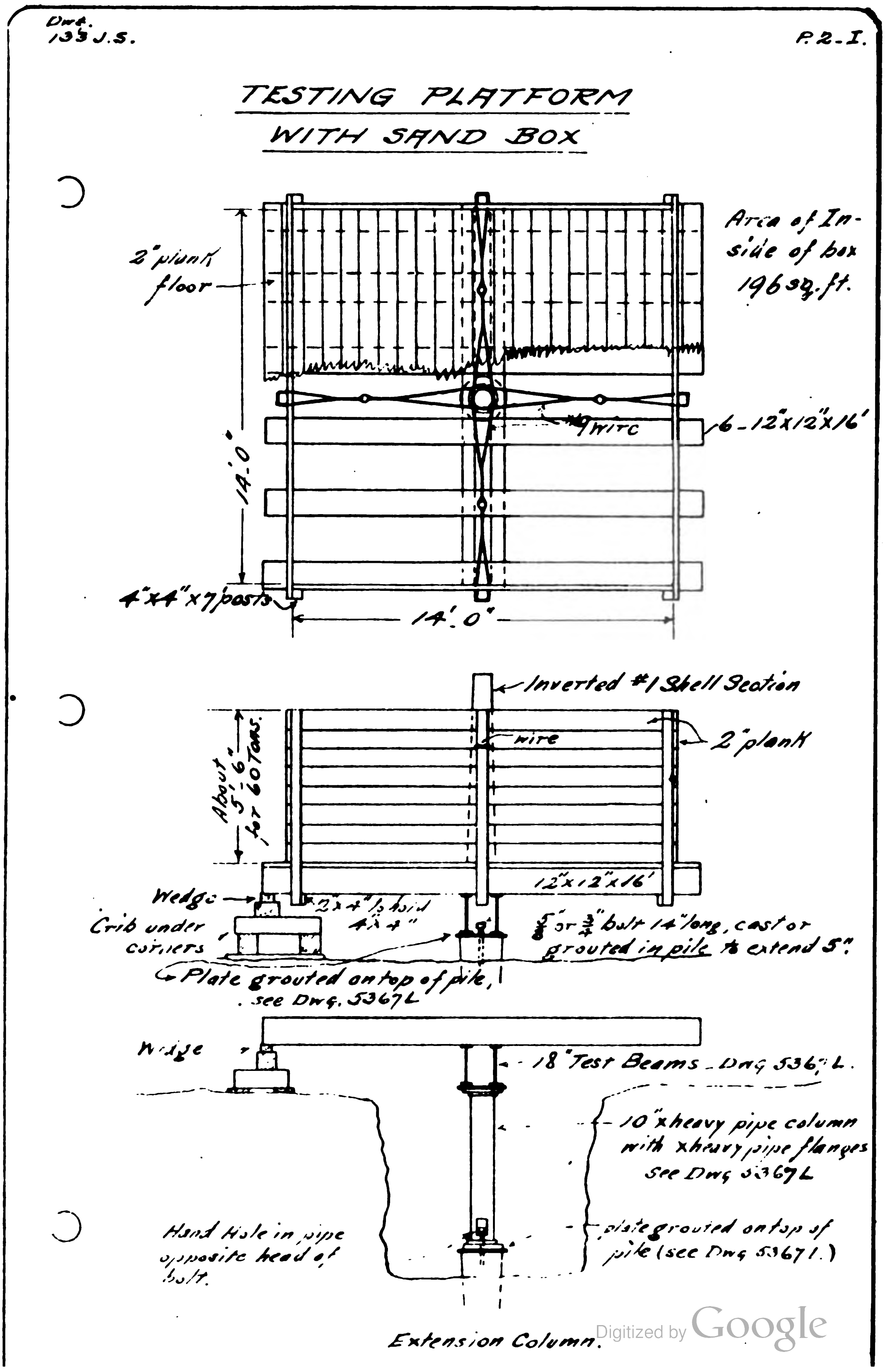
Area on shear line = $8\,3/8" \times 5\,1/4"$

Column 10.

$$\text{Bond stress} = \frac{\text{Stress of column 6}}{p \times \ell}$$

[where p = perimeter of bar and ℓ = length of bar in head]

PLATE XXVI.



Dwg.
133 J.S.
P.2-I.
TESTING PLATFORM
WITH SAND BOX
2" plank floor
Area of Inside of box 196 sq.ft.
#9 wire
6-12"x12"x16'
14'.0"
4"x4"x7 posts
14'.0"
Inverted #1 Shell Section
wire
2" plank
About 5'.6" for 60 Tons.
12"x12"x16'
Wedge
2"x4" to hold 4"x4"
Crib under corners
5/8" or 3/4" bolt 14" long, cast or grouted in pile to extend 5".
Plate grouted on top of pile, see Dwg. 5367L
Wedge
18" Test Beams - Dwg 5367L.
10" x heavy pipe column with x heavy pipe flanges see Dwg 5367L
Hand Hole in pipe opposite head of bolt.
plate grouted on top of pile (see Dwg 5367L.)
Extension Column.

PLATE XXVII.

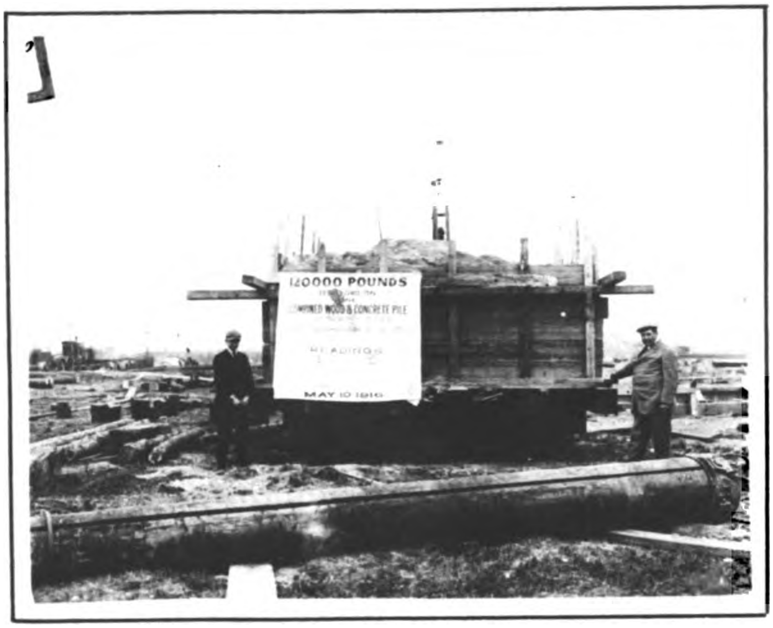


PLATE XXVIII.

PLATE XXIX.

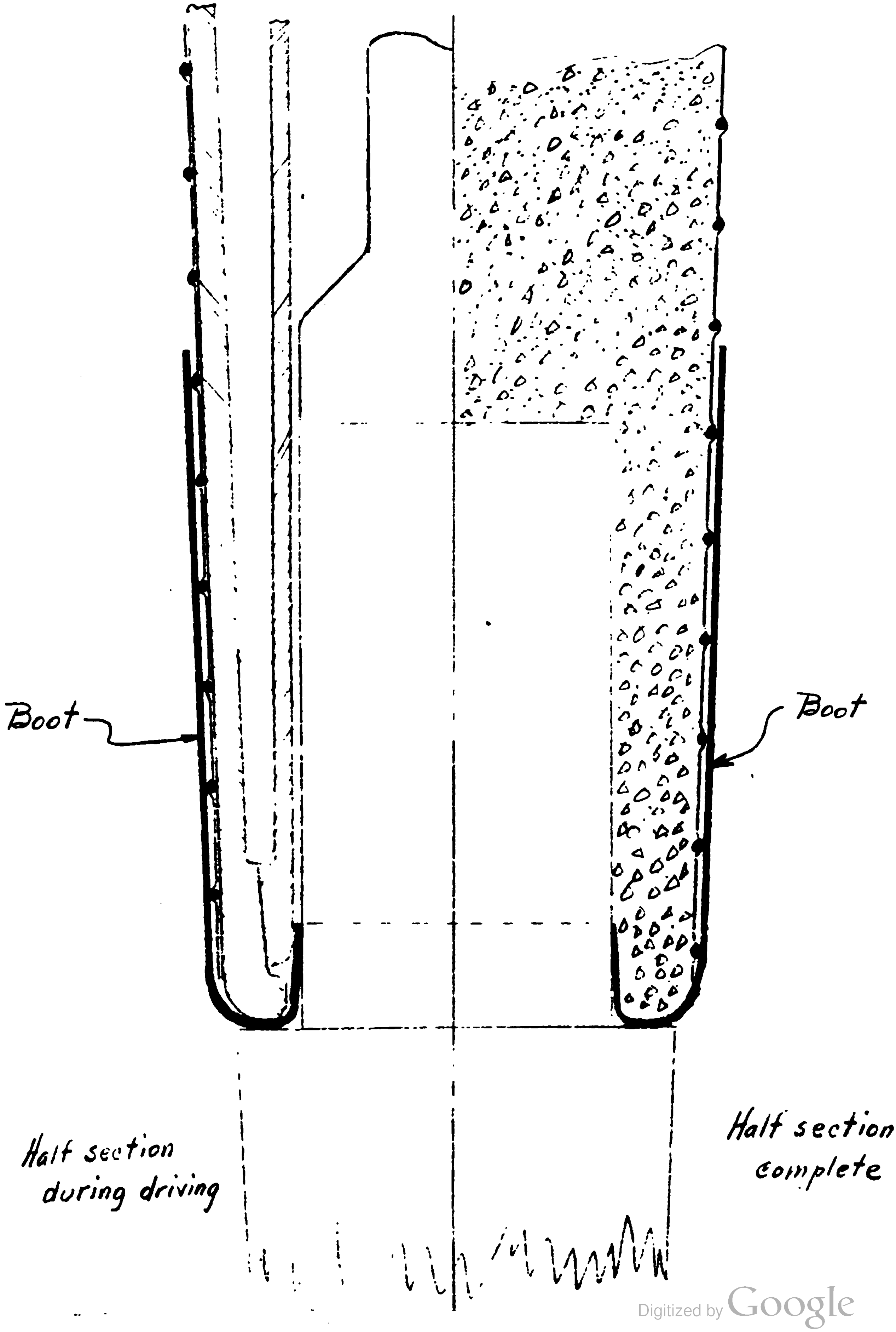

Boot
Boot
Half section during driving
Half section complete

PLATE XXX

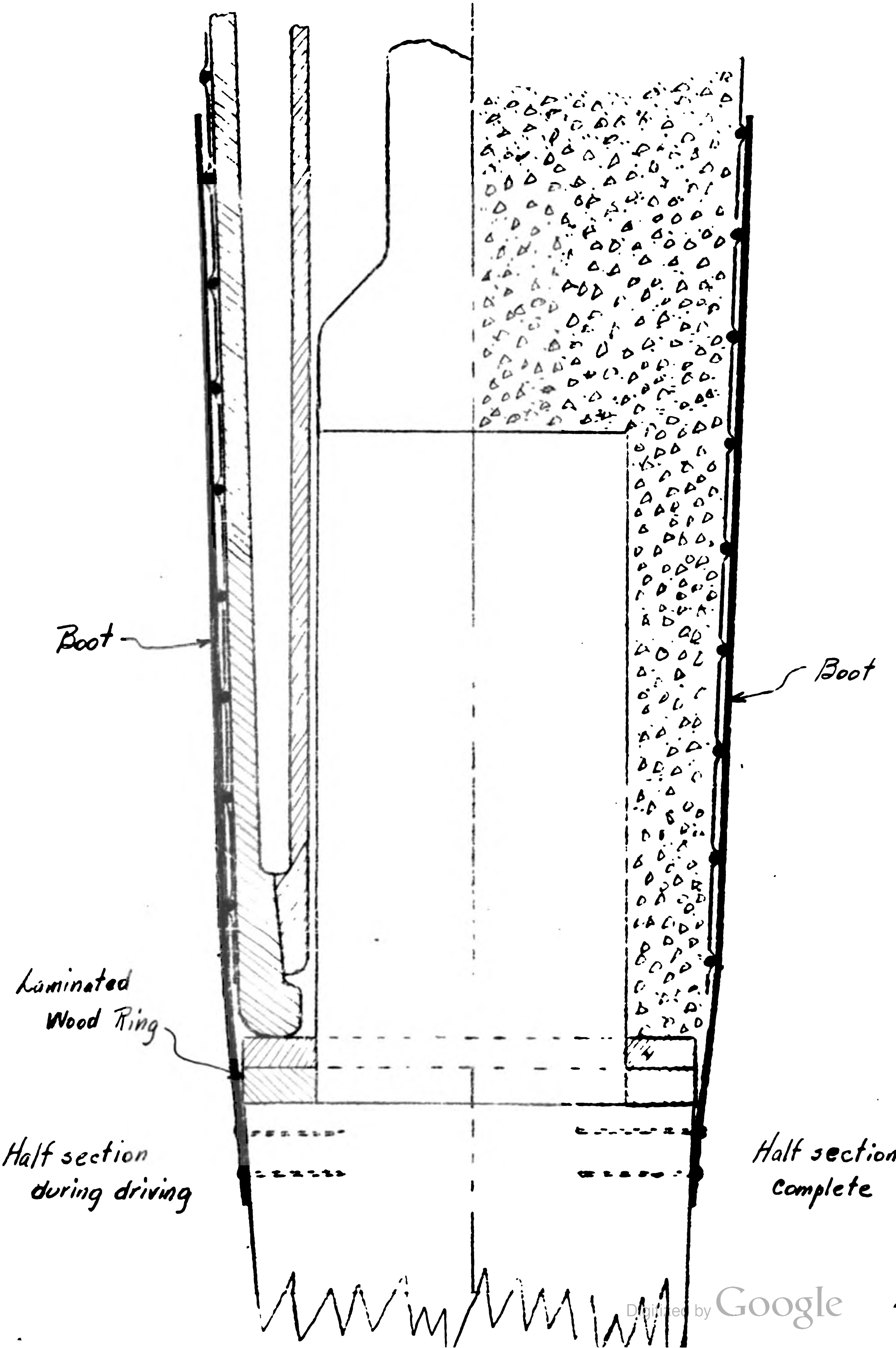
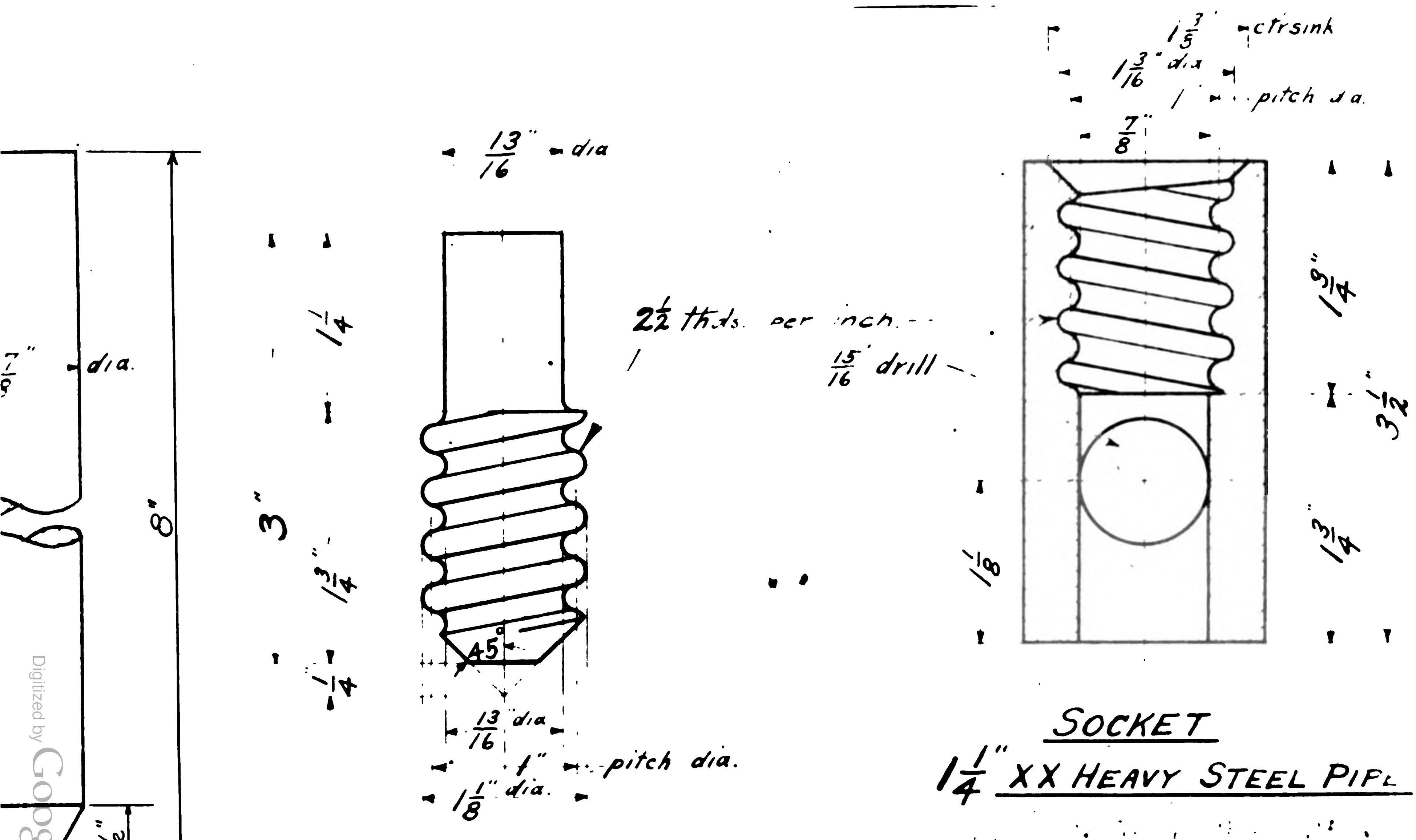

Boot
Boot
Laminated
Wood Ring
Half section
during driving
Half section
Complete

PLATE XXXI



SCREW END.

STEEL.

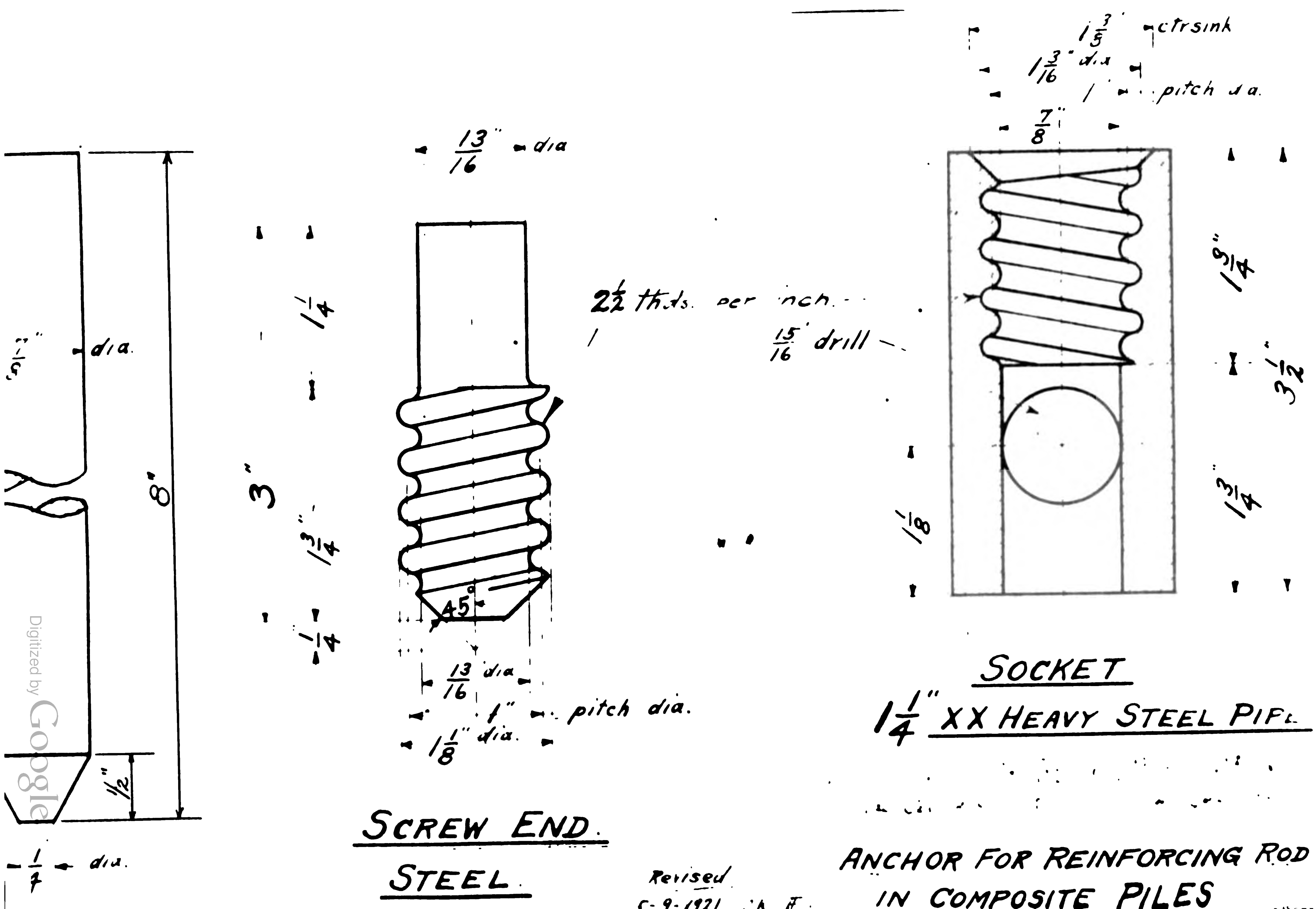
SOCKET

1¼" XX HEAVY STEEL PIPE

ANCHOR FOR REINFORCING ROD IN COMPOSITE PILES

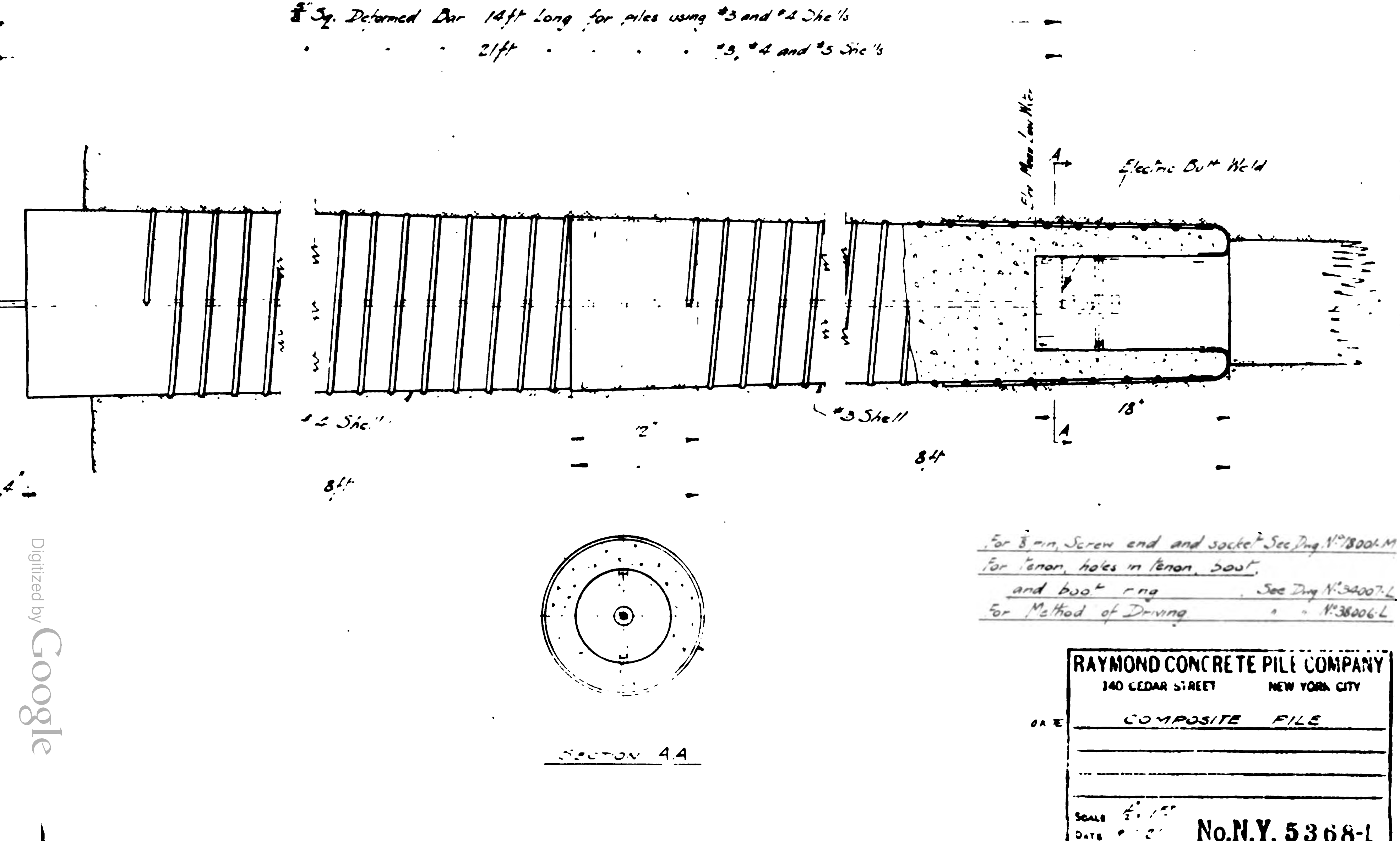
Revised

PLATE XXXI.



SOCKET
$1\frac{1}{4}$" XX HEAVY STEEL PIPE
$1\frac{3}{16}$" dia
pitch dia.
ctrsink
$\frac{7}{8}$"
$2\frac{1}{2}$ thds. per inch
$\frac{15}{16}$" drill
$1\frac{3}{4}$"
$3\frac{1}{2}$"
$1\frac{1}{8}$
SCREW END.
STEEL.
$\frac{13}{16}$" dia
45°
$1\frac{1}{8}$" dia.
$1\frac{1}{4}$
$1\frac{3}{4}$"
$\frac{1}{4}$
3"
8"
$\frac{1}{2}$"
dia.
ANCHOR FOR REINFORCING ROD IN COMPOSITE PILES
Revised

PLATE XXXII.


5/8" Sq. Deformed Bar 14ft Long for piles using #3 and #2 Shells
" " " 21ft " " " " #3, #4 and #5 Shells
Electric Butt Weld
A
A
18"
#3 Shell
#2 Shell
2"
8 ft
8 ft
4"
SECTION A.A
For 3 in. Screw end and socket See Dwg. No. 18004-M
For tenon, holes in tenon, boot,
and boot ring See Dwg. No. 34007-L
For Method of Driving " " No. 38006-L
RAYMOND CONCRETE PILE COMPANY
140 CEDAR STREET NEW YORK CITY
COMPOSITE FILE
SCALE
DATE
No. N.Y. 5368-L

X. APPROVAL

The foregoing thesis is hereby approved as a creditable study of an engineering subject, carried out and presented in a manner sufficiently satisfactory to warrant its acceptance as a prerequisite to the degree for which it has been submitted. It is to be understood that by this approval the undersigned does not necessarily endorse or approve any statement made, opinions expressed, or conclusions drawn therein, but approves the thesis only for the purpose for which it is submitted.

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Professor of Railway Engineering

March 9, 1922